ECONOMICS

of

HIGHWAY BRIDGE TYPES

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PREFACE

This volume has been prepared to meet what the writer feels to be a most urgent need in the field of highway engineering,—a concise discussion (with the necessary cost and quantity data appended) of the fundamentals of economic analysis and type selection for ordinary highway bridge structures.

The text has been compiled with a view to its utility not only to the practicing highway engineer, but to the undergraduate student in highway and civil engineering as well, and it is the writer's thought that such a volume should be assigned to the student prior to or at least simultaneously with his embarkation upon the study of structural design proper.

To the student in engineering, this work should serve a two-fold purpose. First, as an outline of bridge economics and type selection in general, it serves to direct his attention to a most important phase of bridge engineering and one to which scant time is given in most engineering curricula. Second, the many cuts and illustrations, covering in fact practically all of the commonly employed types in highway bridge construction, will serve to give him a rather thorough and comprehensive survey of the ordinary construction types, thus affording him an opportunity to develop the necessary and much to be desired general perspective of the subject before taking up the complicated minutia of design and design methods.

The general discussion of details which forms a large portion of the text of Chapters IV and V should prove of value in calling attention to certain practical points in design, of which sight is very frequently lost.

The tables of quantities and costs should prove of great benefit to the engineer in the field engaged in highway location or relocation, to the highway maintenance engineering organizations and to field parties, divisional organizations, municipal and county engineers and others engaged in the construction, location or design of highway bridges.

Particular attention is directed to the fact that the discussion has been prepared for the highway engineer *in general* rather than the highway *bridge* engineer, carefully avoiding lengthy technical or highly specialized subject matter throughout, and the attempt has been made to make the cost curves and quantity data sheets sufficiently general in application to cover the average practice.

The writer desires to acknowledge the valuable assistance rendered by Mr. W. A. Reeves, Office Engineer for the Oregon State Bridge Department, in editing and correcting the many cost and quantity curves involved.

SALEM, OREGON
FEBRUARY, 1929

C. B. McCullough

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CHAPTER I
INTRODUCTION

The field of highway bridge engineering as a whole may be conveniently considered as composed of the following six distinct major phases or activities:

1. Bridge location.
2. Preliminary investigational work and stream study.
3. Economic analysis and type selection.
4. Detailed design.
5. Construction.
6. Maintenance and operation.

It is with the third group of the above series—"Economic Analysis and Type Selection" that this book has to deal.

It can be said without fear of successful contradiction that this phase of bridge work involves the possibility of saving or of wasting more money than any other engineering detail or operation; in fact, type selection is unquestionably the highest, most difficult and most important feature of bridge engineering from start to finish. Yet, strange as it may seem, although many volumes have been compiled covering the field of design and construction methods *practically no published data are available* concerning this ultra-important phase of the work.

The result has been that the engineering student in attempting to master the many difficult and rather abstruse principles underlying the technique of bridge design has utterly lost his perspective and has failed to grasp the importance of this phase of the problem; in fact, in many instances he has *not grasped the fact that a problem exists at all.*

The practicing engineer also, especially where his work has involved highway bridge design and construction only incidentally, has quite generally failed to appreciate the importance of a correct type selection, with the result that hundreds of thousands, in fact, millions of dollars have been wasted through improper type adaptations resulting in *unwarranted first costs* or needless maintenance expense.

Correct type selection is the very corner stone of economy. A failure to recognize the principles involved or to evaluate correctly the factors entering into the problem may frequently result in a waste many times greater than any saving which may result from refinements in stress analysis and design. It is true that type selection calls for the exercise of the rarest judgment, tempered by long experience in the design, construction, maintenance and operation of bridges under a wide variety of conditions and it is also true that as a general rule, nothing but time will give the bridge engineer the maturity of judgment needed. It is quite possible, however, to analyze this problem, to separate it, as it were, into its component parts, to state certain fundamental principles and submit certain data which may aid in forming judgments as to probable first costs, maintenance costs, renewal costs, etc., for the various construction types commonly employed. Such a treatment involves the preparation of numerous quantity curves involving many bridge types and what is considerably more difficult, the compilation of *actual cost data* as to the annual maintenance expense for various construction types under varying conditions as regards climate, traffic service, etc.

It is the purpose in the chapters which follow to state such underlying principles as may be necessary for a complete understanding of the subject and to submit such data as to quantities and first costs, annual maintenance costs, etc., as are needed for the proper evaluation of the variables introduced in the problem.

Much of these data have been gathered by the writer over a period of fifteen years in the actual design, construction, maintenance and operation of highway bridge structures of various types and while not applicable to every condition or problem, of course, are never-the-less a reasonably safe basis upon which to formulate an economic study.

Particular attention is directed to the discussion of traffic problems presented in Chapter II, a feature of bridge design and type selection which has not heretofore been accorded a discussion commensurate with its general importance. Chapter II does not attempt, of course, to discuss or even state the many traffic features which may enter into any given bridge study, but it does serve to illustrate the type of traffic problems often encountered and serves to call attention to the importance of a thorough study of this phase of each individual selection before going further with the economic analysis.

Attention is also directed to the discussion of "rental values" in Chapter III and the introduction of an offsetting rental charge into the economic equations. This is a comparatively new departure in economic analysis and furnishes a far more rational and just method of type comparison than any other method yet introduced.

CHAPTER II

GENERAL FACTORS CONTROLLING TYPE SELECTION

Article 1.—Introductory

After the preliminary surveys have been completed for any bridge structure *and before* any work can be done on the detailed design or preparation of plans, it becomes necessary to make at least a tentative selection of the type of construction best suited to the particular needs involved. **It is this particular phase of the work, indeed, which is to form the entire subject matter of this volume.**

The question of economy in first cost, maintenance and renewals is naturally a major controlling consideration and one which in order of importance should, possibly, receive first mention. The economics of any bridge problem, however, are generally investigated after certain other controlling conditions have received due consideration, for which reason the question of economics will be reserved for discussion in Chapter III, the purpose of this chapter being to discuss those factors *other than economic considerations* which operate to control type selection.

There are, of course, many considerations which operate to dictate the selection of type for any particular highway bridge. The major controlling factors, however, may be conveniently grouped for discussion as follows:

A. Stream behavior.
B. Requirements of navigation.
C. Traffic considerations.
D. Architectural features and scenic considerations.
E. Condition of available funds.

Each of these will now be introduced and briefly discussed.

Article 2.—Hydraulic Demand or Stream Behavior

The term "stream behavior" is here used to signify the peculiar characteristics of the waterway during periods of high water as regards erosion of bed and banks, lateral shifting of channel, carriage of drift, ice and debris, etc. Such characteristics many times operate to place certain limits upon type selection entirely independent of considerations of economy and it is with such tendencies that this article has to deal.

Before proceeding with the discussion of this phase of the subject, it may be well to point out that in general the entire stream crossing measured from highwater line to highwater line clear across the flood plain is composed of three distinct elements, to-wit:

(a) The main channel structure.

(b) Structural approaches (comparatively short spans).

(c) Approach embankments.

In certain cases it is possible (and if possible it is generally cheaper) to fill out for a short distance rather than to construct a structural approach. It is also possible in many instances to build the approach structure with much shorter individual spans than are needed for the main channel.

With the above in mind, it may now be stated that the particular characteristics of the stream in question may modify the selection of type in the following ways:

(a) By controlling the minimum spacing of piers for the main channel spans.

(b) By controlling the minimum length of main channel construction.

(c) By limiting the points beyond which an approach embankment may not, in safety, be placed.

(d) By controlling the length of individual approach spans and type of approach construction.

(e) By limiting the net vertical clearance requirements.

Taking these up in order, we may first consider the question of pier spacing over the main channel.

A.—**Minimum Spacing of Piers.** The minimum distance between piers may be limited by the necessity for

(a) Reducing to a predetermined amount the backwater head during extreme flood flow.

(b) Keeping flood currents below a certain critical velocity because of erosive tendencies.

(c) Protection against ice, drift or debris.

Doubtless this is not the place for an extended discussion of the hydraulics of stream flow; however, the following roughly approximate formulas serve to illustrate the effect of piers and obstructions in any river channel in general.

If we let

Q=The discharge of any stream in second feet through the main channel

A=The area of the main channel waterway

V=The mean velocity through the main channel

then

$$Q=VA$$

The effect of the placement of any pier is to reduce the value of A with the result that for a given flood discharge V is correspondingly increased.

Now V, the velocity, is in turn a function of the slope of the stream and in order that V may be increased sufficiently to carry the requisite discharge, the slope of the flood crest must be increased as the water passes between the piers.

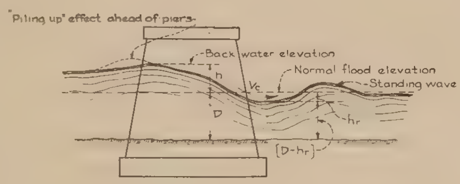


Figure 1
Typical flood profile thru constricted channel opening

Figure 1 indicates the general way in which a river pier obstructs the free flow of the stream and backs up the water in the main river channel.

The derivation of hydraulic formulas for various conditions of stream flow is quite a complicated matter and need not be given at this point.

The effect of the pier obstruction, however, may be roughly determined from the following approximate formulas.

Let

h=the back water head in feet

g=the acceleration of gravity=32.16 ft. per sec.

V=the mean velocity through the main channel if no piers exist

V'=the increased velocity caused by the pier obstruction

W=the aggregate *clear* width between extreme channel piers

D=the average depth of the water between piers

C=a constant depending upon the shape of the pier

Then

$$h=\frac{1}{2g}\left[\frac{Q^2}{(CWD)^2}-V^2\right]$$

and

$$V' = \sqrt{V^2 + 2gh}$$

"C" will vary from 85% for square ended piers to about 92% for piers with rounded noses on both ends.

The above formulas are not exact, but are sufficient for this purpose, once the main channel flood discharge has been determined.

It will be observed that unless the channel obstruction is very great, the back water head is relatively small, so that it is not always necessary to investigate the back water effect in any great detail. There are, however, many instances wherein the bridge structure is so located as regards adjacent property as to render even a few inches of additional back water a very serious matter. For example, an adjacent dyke or revetment may be topped or a railway track flooded or perhaps expensive landscaping may be inundated by a very small amount of additional water stage. In such cases, it becomes extremely important to select the spacing of piers and the length of the main structure in such a way as to minimize property damage.

The increase in channel current velocity resulting from the obstruction to flow, caused by the piers, is much more marked than the backwater head, but whether or not such velocity increase is an objectionable feature will depend to a large extent upon the character of the stream bed and the "channel control." Certain streams are said to have good "channel control" by which is meant that the bottom of the channel does not shift perceptibly in depth during the seasonal variation in water stage and current velocity, while other streams (like the Colorado River, for example) having very soft bottoms and a wide fluctuation in stage, have a stream bed which is virtually *in constant motion*. As the stream rises, the increased bottom currents erode and pick up the bed material, dropping it as the flood crest recedes. For conditions of this kind, the bed of the stream may erode downward as much as two feet for every foot rise in the water stage.

For streams of the former class, local erosion at piers, due to increased flood currents, is not greatly to be feared, while for streams of the latter class "pot hole" formation around piers may prove a serious menace and the velocity of flood currents can not safely be increased to a value much above normal. For stream beds of this latter class, therefore, pier spacing should be kept very liberal, unless exceptionally deep foundations are provided.

For shallow streams, soft silt bottoms will generally erode at a bottom velocity of less than one-half foot per second, sand bottoms at a velocity of from 1 to 1.5 feet per second, ordinary clay at a velocity of about 2 to 3

feet per second, compact clay at a velocity of from 5 to 6 feet per second and heavy gravel at a velocity of from 4 to 6 or 7 feet per second. It must be observed that even though piling be used for foundations in strata of this kind, localized erosion around piers may operate to expose the piling and destroy the lateral stability of the structure, even though the vertical bearing resistance of the piles is ample to support the load which furnishes still another reason for keeping the channel free from pier obstruction to the maximum possible extent.

Another local condition which affects the selection of pier spacing is the danger of current deflection against adjacent banks thus endangering property below the structure. This, of course, is a local condition and must be studied locally.

Ice and drift effects also have an important bearing upon pier spacing. The length and character of drift has much to do with the extent to which provision for the same should be made in the design; where long logs and heavy trees are apt to come down on each flood crest the piers should be far enough apart to carry the average tree crosswise of the channel.

The extent of ice damage is dependent to a large extent upon the probability of gorge formation and this, in turn, is a function of the alignment of the stream at the site and other local conditions which must, of course, be studied locally.

In closing, it should be pointed out that it is not possible to discuss or to even anticipate the many ways in which the particular characteristics of the waterway will effect the safe minimum spacing of piers, nor to evaluate many of the various factors involved. It is not the purpose of this discussion to attempt any such evaluation, but rather to call attention to the need for a careful study of each individual case.

In general, it may be said that where back water head, due to pier and approach obstruction may in any degree prove a controlling factor the *hydraulics* of the problem should receive a thorough analysis solving the various formulas for each tentative span arrangement and limiting the final selection to those types which keep the back water head a minimum.

In all cases, a careful investigation should be made of other existing structures along the same stream above and below the site in question and the limiting spacing of piers determined in accordance with the best judgment of the engineer, which judgment should be controlled by a consideration of the underlying principles of stream flow hereinabove set forth.

B.—**Limiting Length of Main Channel Structure.** So far, the discussion has dealt with the spacing of piers for that portion of the structure over the main channel. The next problem is that of determining how far out on the banks this type of construction must extend. Where the stream is entirely confined to one well defined channel, this problem, of course, does not apply, but most streams flow in comparatively broad flood plains over which approach construction having much shorter individual span lengths is a possibility to consider.

There are two types of streams of this latter class; one wherein the main channel is laterally stable and another known as the "meandering stream," where the main channel presents a tendency to shift laterally from side to side of the flood plain. A general discussion of lateral stream movement was given by the writer in a bulletin dealing with highway bridge location and printed by the U. S. Department of Agriculture (Bulletin 1486) to which reference is made for a more extended treatment of this subject. Suffice it to say that where the stream shows a marked tendency to shift laterally in its channel, over a period of years, it may become necessary to carry the main channel construction clear across the flood plain where otherwise, short span approach construction would be entirely adequate and much more economical. In such a case, of course, it may be possible through the use of revetment work, bank protection, dykes or wing dams, to stabilize the channel and the cost of such work should be estimated and balanced against the additional cost of long span construction over the entire flood plain.

C. & D.—**Extent of Approach Embankment Construction and Length of Individual Spans in Approaches.** The type of construction over the flood plain presents the same problems as regards selection as that for the main channel portion of the structure. The hydraulic capacity, that is to say, the need for keeping the value of A, the waterway area, large enough to avoid undue channel currents or back water head will limit the extent to which the approaches may be filled. In this connection it may be well to point out that the mean velocity over the flood plain is, in general, much less than that for the main channel. If it is desired to consider filling a certain portion of the flood plain having an area A'' and a mean flood velocity of V'' feet per second (which velocity must, of course, be determined by current meter readings or by means of Kutter's formula for every portion of the entire waterway cross-section), then

The amount of reduction in discharge capacity will be represented by the term

$$Q''=V''A''$$

The velocity in the main channel will be increased roughly to the value given by the formula:

$$V' = V + \frac{Q''}{A}$$

where V and A refer to the main channel and the water stage in the main channel will be raised by an amount h where

$$h = \frac{1}{2g}\left(V'^2 - V^2\right)$$

Again it may be stated that these formulas are very rough, but serve to give a general idea of the effect of filling any portion of the flood plain.

In addition to the above there is also the question of lateral erosion as the water turns and parallels the embankment in getting back to the main channel. Local bye-channels and overflow passes may necessitate the extension of the structural approach to avoid bank cutting, etc. These, and many other like features are manifestly local problems and must be studied locally.

The structural approach proper should be of sufficient span length to carry the drift and debris which will in general be small at this point and to avoid erosion of pedestals or foundations. A study of the ground will give a very good indication of the strength of flood currents at different points along the flood plain. Heavy brush and willows are evidence of a sluggish current, fine silt and sand of a more rapid current and heavy gravel of a marked current wash. The type of bridge foundations (whether pile or pedestals) and the length of individual spans can be determined from these indications and from the drift.

For ordinary flood plain conditions approach spans of from 15 feet to 20 feet center to center of supports will give very little trouble if foundations are properly placed as to depth.

Certain portions of the flood plain may require longer span construction as plate girder, I-beam or reinforced concrete viaduct types. These problems again are largely local and must be solved to meet the individual case.

E.—**Limiting Vertical Clearances.** Generally the vertical clearance minimum is dictated by the necessity for safety from drift or ice although extreme flood water elevation alone may at times be the limiting factor. In general the superstructure should clear the largest tree root likely to pass the structure or the highest probable ice jam crest by a margin of several feet, if this is at all possible.

The question of vertical clearance may eliminate the consideration of deck truss construction, even though the latter type is, in every other way, the most desirable selection.

In cases where deck trusses of ordinary depth will not afford the requisite high water clearance, it is oftimes necessary to balance the cost of special shallow truss construction, such as will afford this clearance, against the cost of raising the general grade which latter operation, even if feasible, is apt to throw considerable additional expense into the approaches.

Article 3.—Requirements of Navigation

Considerations of this kind will generally affect type selection, as regards both vertical and horizontal clearances for the main channel span. Where movable spans are used, the *type of design* for the moving leaves may also be controlled by considerations of water traffic. In many of the streams in this country, not only the main channel span, but also the flanking spans are held to a certain established minimum as regards vertical and horizontal clearances by the United States War Department, so that this fact may control the spacing of piers, regardless of other considerations.

In many cases where War Department restrictions limit the *minimum length of any single span* between harbor lines, it becomes necessary to increase even this minimum in order to make the pier spacing come out even at the harbor lines and thus avoid the introduction of a short end span.

On certain navigable waterways, channel control is difficult and constant dredging operations must be maintained. In cases of this kind the necessity for keeping a free and unobstructed waterway, in order to prevent shoaling and bar formation below the bridge site, may render it necessary to carry approach trestle or viaduct construction in place of a "mole" or filled approach even though the latter is in every other way desirable and economical.

In the matter of choice between fill versus trestle, the following quoted from the regulations of the War Department in a publication entitled "Laws for the Protection and Preservation of the Navigable Waters of the United States" is of interest, as it prescribes the procedure under which a fill may be extended beyond any established harbor line.

"That the creation of any obstruction, not affirmatively authorized by Congress, to the navigable capacity of any of the waters of the United States is hereby prohibited; and it shall not be lawful to build or commence the building of any wharf, pier, dolphin, boom, weir, breakwater,

bulkhead, jetty, or other structure in any port, roadstead, haven, harbor, canal, navigable river, or other water of the United States, outside established harbor lines, or where no harbor lines have been established, except on plans recommended by the Chief of Engineers and authorized by the Secretary of War; and it shall not be lawful to excavate or fill, or in any manner to alter or modify the course, location, condition or capacity of, any port, roadstead, haven, harbor, canal, lake, harbor of refuge, or inclosure within the limits of any breakwater, or of the channel of any navigable water of the United States, unless the work has been recommended by the Chief of Engineers and authorized by the Secretary of War prior to beginning the same.

"That where it is made manifest to the Secretary of War that the establishment of harbor lines is essential to the preservation and protection of harbors he may, and is hereby, authorized to cause such lines to be established, beyond which no piers, wharves, bulkheads, or other works shall be extended or deposits made, except under such regulations as may be prescribed from time to time by him: PROVIDED, That whenever the Secretary of War grants to any person or persons permission to extend piers, wharves, bulkheads, or other works, or to make deposits in any tidal harbor or river of the United States beyond any harbor lines established under authority of the United States, he shall cause to be ascertained the amount of tide-water displaced by any such structure or by any such deposits, and he shall, if he deem it necessary, require the parties to whom the permission is given to make compensation for such displacement, either by excavating in some part of the harbor, including tide-water channels between high and low water mark, to such an extent as to create a basin for as much tide water as may be displaced by such structure or by such deposits, or in any other mode that may be satisfactory to him."

All of the above questions may be settled by taking up the matter in advance with the office of the nearest U. S. District Engineer Officer.

In connection with the bridging of navigable waters of the United States, in general it should be noted that the War Department has the right to demand at any time that an existing bridge (even though the same has been constructed under an approved plan) be altered, changed or removed entirely, if the same constitutes what may be considered an *unreasonable obstruction to navigation.* The following paragraph quoted from the compilation of laws mentioned hereinabove covers this regulation:

"That whenever the Secretary of War shall have good reason to believe that any railroad or other bridge now constructed, or which may hereafter be constructed, over any of the navigable waters of the United States is an unreasonable obstruction to the free navigation of such

waters on account of insufficient height, width of span, or otherwise, or where there is difficulty in passing the draw opening or the draw span of such bridge by rafts, steamboats, or other water craft, it shall be the duty of the said Secretary, first giving the parties reasonable opportunity to be heard, to give notice to the persons or corporations owning or controlling such bridge so to alter the same as to render navigation through or under it reasonably free, easy and unobstructed; and in giving such notice he shall specify the changes recommended by the Chief of Engineers that are required to be made, and shall prescribe in each case a reasonable time in which to make them. If at the end of such time the alteration has not been made, the Secretary of War shall forthwith notify the United States District Attorney for the district in which such bridge is situated, to the end that the criminal proceedings hereinafter mentioned may be taken. If the persons, corporations, or association owning or controlling any railroad or other bridge shall, after receiving notice to that effect, as hereinbefore required, from the Secretary of War, and within the time prescribed by him, willfully fail or refuse to remove the same or to comply with the lawful order of the Secretary of War in the premises, such persons, corporation, or association shall be deemed guilty of a misdemeanor and on conviction thereof shall be punished by a fine not exceeding five thousand dollars, and every month such persons, corporation, or association shall remain in default in respect to the removal or alteration of such bridge shall be deemed a new offense, and subject the persons, corporation, or association so offending to the penalties above prescribed: PROVIDED, That in any case arising under the provisions of the section an appeal or writ of error may be taken from the district courts or from the existing circuit courts direct to the Supreme Court either by the United States or by the defendants."

In the selection of type, therefore, the bridge engineer should be interested not only in getting his application for permit approved, but also to see that every portion of the construction is, and will be at all times, truly adequate for navigation needs.

Article 4.—Traffic Considerations

Chief among considerations of this character which control type selection, may be mentioned the following:

A.—**Sight Distance.** Figure 2 illustrates the sight restriction due to the employment of housed timber truss construction on an alignment having sharp curvature approaching the span. Through steel truss construction is open to this same objection, but naturally, to a very much less extent. With present day traffic concentration, highway bridges should

Figure 2. Restricted sight distance for covered span on sharp curved approach.

be designed with the greatest possible regard to clear and unobstructed vision.

B.—**Traffic Movement.** Traffic service is the *ultimate purpose* of any bridge structure. In the question of type selection, therefore, thorough

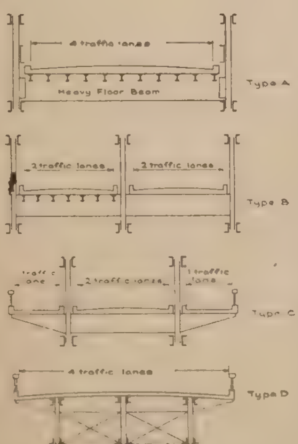


Figure 3

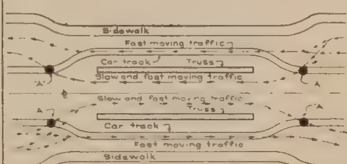


Figure 4b

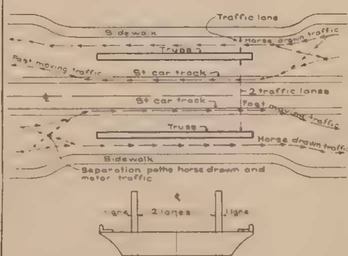


Figure 4a

consideration should be given to such questions as (a) the *direction of traffic movement* over the span, (b) provision for the *separation of fast and slow moving traffic,* (c) *distribution and collection of traffic with minimum congestion at the bridge head,* etc. Traffic roadways should be designed to accommodate the maximum existing and probable future traffic needs, and where traffic density is great ample roadway width must be provided. Where the traffic demand necessitates a wide roadway a condition such as is illustrated in Figure 3 may be found to exist. Type A employs a two truss design, but the floor system becomes so heavy as to prove uneconomical. Type B is much cheaper in first cost, but involves a separation of traffic lanes. This condition is not particularly objectionable, if traffic density in both directions is balanced at all times. In the majority of cases, however, this balance does not exist. If, for example, eastward traffic is heavy in the morning and westward traffic heavy in the evening, an unobstructed roadway will permit three lines of traffic eastward to one westward in the morning. In the evening conditions will automatically reverse themselves, thus affording the necessary flexibility in capacity. If a truss is placed in the center of the roadway, however, traffic in either direction is limited to a two lane way, regardless of the relative densities. An arrangement designated as Type C (Figure 3) is sometimes employed. Such an arrangement leads to serious complications, especially in connection with street car or interurban traffic as will be seen by a study of Figure 4. If the arrangement (shown at 4a) is employed wherein street car tracks run straight through between the trusses, slow moving traffic (horse drawn) must be placed outside the trusses, as otherwise such traffic will seriously obstruct street car movement within the span. Such slow moving traffic has the effect of forcing all other traffic to the center of the bridge on account of the absence of adequate passing space outside the truss. This practically reduces the effective capacity of the structure to a two lane roadway, as far as motor traffic is concerned. If, on the other hand, the arrangement shown in Figure 4b is employed, fast traffic can occupy any portion of the roadway, as there is adequate passing space inside the truss. However, this arrangement introduces *four* crossing points between vehicular and street car traffic lines (two at each end of the bridge, as shown at the points designated "A" in Figure 4). This condition is obviously highly objectionable, especially in wet weather.

The deck truss (Type D, Figure 3) eliminates every objection above listed and from a traffic standpoint is much superior to any other type. It may be that clearance requirements eliminate this type from consideration on the proposed grade line, in which case it becomes necessary to investigate the feasibility and cost of a new grade line modified so as

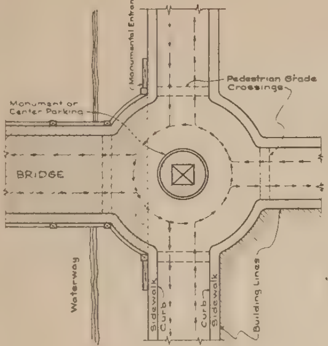


Figure 5
STANDARD TWO LANE TRAFFIC PLAZA

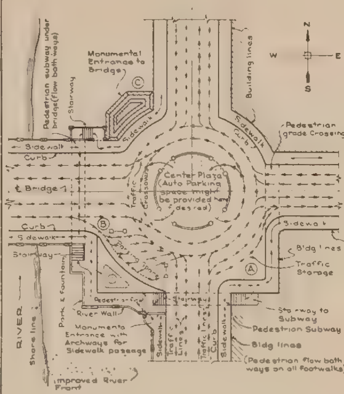


Figure 6
LARGE TRAFFIC PLAZA TYPES

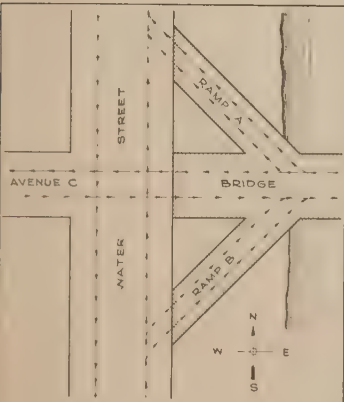


Figure 7

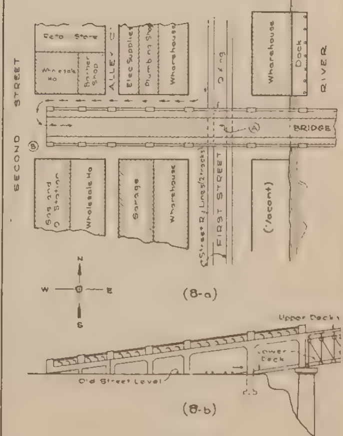


Figure 8

to permit deck construction and also the feasibility of a special shallow truss design. In the final analysis when the relative economy of the different types is determined, the unobstructed roadway should be given a great deal of preference and should be chosen unless cost considerations are prohibitive or other reasons make its use impracticable.

C.—**Traffic Collection and Distribution.** By virtue of its position, a bridge structure, especially when located in or adjacent to a municipality, is a point of traffic constriction, the "neck of the bottle," as it were, and in type selection as well as in location some consideration must be given to the provision of adequate collection and distribution facilities. Such facilities may in general be divided up into two main subdivisions:

(a) Traffic plazas (in general located outside the limits of the structure itself).

(b) Ramps or bilateral approaches constructed as part of the bridge structure.

A typical traffic plaza suitable for a two lane traffic way is illustrated in Figure 5. It will be observed that all vehicular traffic intersections are eliminated, all traffic being picked up on the circle, rotated to its outlet and distributed, traffic along the circle thus moving *in one direction only*. It will be noted, however, that the pedestrian crossings involve an intersection with vehicular traffic lines. This condition can be eliminated by adopting a system of pedestrian subways, as described later on. Figure 6 illustrates a traffic plaza arrangement suitable for heavier traffic densities. In this arrangement all traffic is delivered to a distribution circle containing two or more traffic lanes. All traffic delivery is made on the outside lane and works its way to the inner lanes through a system of "cross over" paths, the inner paths being used only in times of heavy traffic flow. From the plaza circle traffic delivery to the various outlets is also made from the outer lane of the circle, traffic working its way to this outer circle before reaching its point of delivery by means of a system of oppositely directioned "cross overs" as shown in the figure. It is possible for traffic lines to cross in this arrangement, but ordinarily the traffic will swing to the outer circle in sufficient time to avoid any direct intersection of lines. The pedestrian traffic may be taken care of at grade or by means of foot traffic subways, as shown in Figure 6. Under the bridge proper it is generally a comparatively easy matter to arrange for a pedestrian footway. Under the graded streets the cost of a pedestrian subway is greater, but by no means prohibitive. Where pedestrian traffic crosses the streets at grade, an arrangement known as "traffic parking storage" may be made, as shown at point "A" in Figure 6. Here traffic heading north or west will continue around

the circle, while traffic headed east, when stopped by the pedestrian traffic north or south along the street crossing, may pull into the "traffic storage" space at "A" (thus being out of the way of the through traffic) and wait until pedestrian traffic has cleared. If the bridge head is so located as to create a demand for parking facilities for autos, the same may be provided at the entrance, as shown at point "B". The center plaza or circle may also be converted into parking space, if necessary, although this can hardly be regarded as a desirable arrangement. In general traffic plazas are designed with some architectural significance and the core of the circle parked or marked with a suitable monument. Figure 6 illustrates four different corner treatments which are typical of traffic plaza arrangements of this character. In the case of memorial bridges, a rather dignified and pleasing effect may be secured through the adoption of a monumental entrance treatment. In the style of treatment shown at "B" the sidewalk passes through an arch way in the entrance monument, at point "C" is indicated an arrangement wherein the sidewalk passes in front of the entrance structure. The type of traffic plaza and entrance treatment to be adopted in any case is dependent upon many local factors and the general discussion need be carried no further at this point. In any case, however, the arrangement employed should be based upon a study of the general trend of traffic flow, the maximum flow being given the maximum channel.

"Ramps" or "bilateral approaches" are traffic outlets or intakes placed upon the structure itself or constructed as a component part of the same. From Figure 7, which indicates a typical double ramp arrangement, it will be observed that the number of traffic line intersections is not reduced through the employment of the ramps or bilaterals. The rapidity of traffic movement to and from the bridge is, however, greatly increased. When traffic is open on Water Street, all north bound traffic on the bridge would be held up were it not for ramp "A", which permits the above traffic to move off the span, thus relieving the bridge congestion. During this same period ramp "B" permits east bound traffic on Water Street to head into the bridge, thus helping to clear Water Street more quickly than otherwise.

D.—**Traffic To and From Waterfronts.** Where the highway bridge crossing is within the corporate limits of a city or town, it frequently happens that the bridge structure must be designed with due regard to the provision for traffic to and away from the water front, and also for traffic from the business places facing the bridge structure on either side of the street, as shown in Figure 8. Questions of such character may modify the type selection, both as regards span lengths

for approach construction, roadway widths on approaches, and location of grade line. For navigable stream crossings such considerations may also effect the choice between high level vs. movable span construction. In view of the above, therefore, a general discussion of some of the major traffic problems involved, together with a few typical solutions, may be of value at this point.

Consider for example, the general layout shown in Figure 8. The street marked "First Street" is a heavy hauling traffic way along the river front and contains a double track street railway line. The street upon which the bridge structure is located is occupied by shops and wholesale stores. Alley "C" is rather active from a traffic standpoint and must be kept open. The traffic problem resolves itself into the following:

(a) *Provision for a grade separation at point "A".*

This is extremely important, otherwise there arises a condition wherein through traffic moving at a comparatively high speed is intersected with slow heavy hauling traffic, moreover a certain portion of this heavy traffic may turn eastward across the bridge, which would operate to congest traffic at the bridge head were the grade line lowered to intersect point "A" at the grade of First Street. In addition to the foregoing, considerations of traffic safety demand a separation of grades at all intersections of highway and railway traffic wherever possible. In this case, the railway traffic is city electric service, but even in this case the element of danger is only lessened and not by any means eliminated.

(b) *Provision for traffic from points on First Street eastward across the bridge.*

With the high grade line necessary for a grade separation at point "A", any traffic originating on First Street and having a destination eastward across the bridge will be compelled to travel a very devious and lengthy path and to make a sharp turn at point "B" to get onto the bridge, as shown by the arrow path (Figure 8a). This is objectionable from the standpoint of local traffic itself and even more so from the standpoint of through traffic, as it introduces the very element of congestion at the bridge head that the grade separation has sought to eliminate.

Among remedies which suggest themselves in this case are the following:

(1). A double deck design routing the heavy traffic directly onto the lower deck. The feasibility of this procedure would, of course, depend upon the limiting vertical clearance requirements for the river spans. If this were to be done, all heavy hauling should be routed so as to reach

the lower deck, thus leaving the upper deck free for fast moving traffic only. A traffic routing of this kind might necessitate a wider roadway along the side of the bridge approach between First and Second Streets than would otherwise be needed.

(2). The construction of a series of ramps to connect First Street with the bridge. This construction might perhaps involve a greater expense than the first solution and would not separate fast and slow moving traffic. Figure 9 indicates a double track single ramp connection to First Street. If conditions warranted the expenditure, two single-traffic ramps, one leading north and the other south, as indicated by the dotted lines in Figure 9 might be considered.

(3). The construction of a heavy traffic or truck elevator or vertical transfer table, as shown in Figure 10.

(4). A spiral approach from First Street to the bridge.

(c) *Provision for traffic egress and ingress for the business places facing the bridge.*

From a consideration of Figure 8 it will be observed that the presence of the bridge structure results in a restriction of roadway width adjacent to the property facing the same. When the residual roadway width is so small as to seriously obstruct traffic, special consideration must be given to this phase of the problem. The following solutions have been adopted in certain instances in past practice:

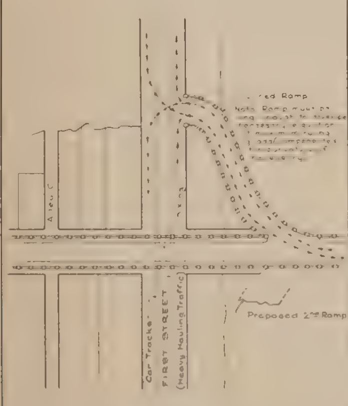


Figure 9
TYPICAL DOUBLE TRACK RAMP OR BY-LATERAL CONNECTION FOR PURPOSE OF CONNECTING FIRST ST WITH THE BRIDGE DECK LEVEL

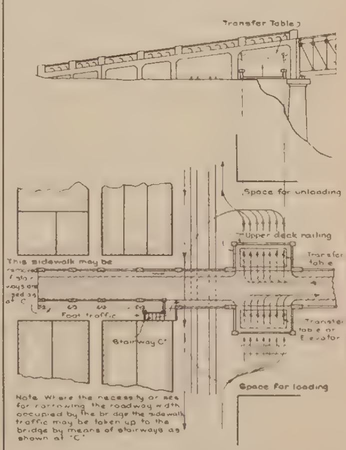


Figure 10
TRUCK ELEVATORS OR VERTICAL TRANSFER TABLES

(1). The widening of the approach roadway to extend clear across the street (see Figure 11a). This procedure involves a remodelling of the fronting buildings and converts a portion of each lower story into basement area. The expense involved is generally very large, but the general effect is good.

(2). The use of "Arcade" work to afford greater traffic way is shown in Figures 11a and 11b. Generally the arcade width is just sufficient for the foot walk. However, a double traffic way and foot walk may easily be provided in this manner, as indicated in Figure 11b.

(3). The removal of sidewalks from the approach bridge structure throughout a portion of the length of street frontage and the use of stairways or inclined ramps for connecting sidewalk traffic with the bridge (see Figure 10). This arrangement has been used in certain

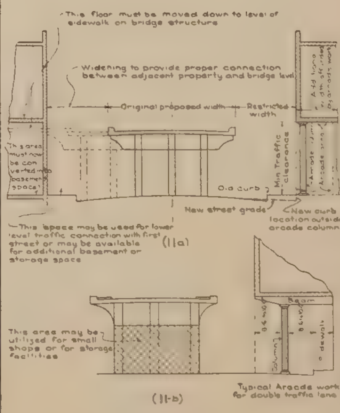


Figure 11

cases, but is open to the objection that pedestrians do not like to climb stairs and in general will take the roadway up to the main bridge structure, even though no sidewalks are provided, thus introducing an element of traffic danger.

Any of the above methods involve a considerable outlay of expense. In certain cases this expense has been offset, to a certain extent at least, by converting the area under the bridge approach into small shop or storage rooms, as shown in Figure 11b. In certain districts the rental

value of such space is considerable. It should be noted, however, that as a general rule it is desirable that the approach structure be left open in order to afford better lighting and vision for the abutting business places. In fact, in many instances viaduct construction is employed for this purpose, even though retaining walls and filled approaches would be much cheaper. In the case shown in Figure 8 alley "C" should be kept open in any event.

The several points heretofore discussed are but a very few of the many traffic problems which may operate to control type selection; in fact, even the briefest possible treatment of all such problems would fill a volume in itself. There is, for example, the question of pedestrian traffic, its rate of speed under varying conditions and provision therefor, the question of moving walks, escalators and stairways, shuttle car service, arrangement of car tracks in reference to traffic plazas, comfort stations, methods of crossing vehicular and pedestrian traffic, parking storage, and a thousand and one other items which go to make up the traffic features of any bridge study.

The point to be made in connection with this discussion is simply that all such features operate to place certain limits on *type selection*, gener-

Figure 12. The Robert A. Booth Memorial Bridge at Winchester, Oregon, illustrating the use of Gothic arches, curved piers, and a dentil and bracket treatment.

ally in regard to the spacing of piers, arrangement of trusses or girders, arrangement of sidewalks, type of approach construction, choice between through and deck construction, grades, clearances, roadway widths, etc.

All such matters, therefore, must receive careful and individual consideration before the economics of the problem can be applied.

Article 5.—Architectural Features and Scenic Considerations

In certain cases, considerations of this character may play an important part in dictating type selection. Grouped in the order of their architectural possibilities bridge types may be classified, as follows:

(a) Masonry arch construction.
(b) Reinforced concrete deck construction.
(c) Deck truss or plate girder construction with concrete deck and railing.
(d) Reinforced concrete through girder construction.
(e) Through truss or girder construction.
(f) Timber construction.

The masonry arch lends itself readily to excellent architectural inter-

Figure 13. Construction view of a reinforced concrete viaduct type with arched curtain walls and panel added for architectural effect.

pretation. Figure 12 is an illustration of a rather pleasing treatment, this being a view of the "Robert A. Booth" Memorial Bridge at Winchester, Oregon. Figures 88, 89 and 91 of Chapter IV are also illustrative of the architectural possibilities in this type of design. Deck truss or girder construction with concrete floor and rail is also susceptible of a rather pleas-

ing treatment, as shown in Figure 84 of Chapter IV while Figure 13 is a construction view of a concrete girder viaduct type, wherein arched curtain walls are employed for architectural effect. In type selection for architectural effect, consideration should be given to the degree to which the structure will be exposed to view. If the alignment is such that the structure is plainly visible in side elevation from the approaching highway, more attention should naturally be paid to a type selection which gives a pleasing side elevation outline than if only the roadway were ordinarily visible. Furthermore, the natural scenic setting will play a large part in determining the extent to which considerations of aesthetics should be allowed to influence type selection. The general nature of the highway, the location of the bridge in reference to parks, pleasure resorts, etc., are also determining factors in this connection.

Article 6.—Condition of Available Funds

In certain instances the choice between types may hinge upon the amount of money actually available for construction purposes and thus render of no avail any theoretical consideration of the economics of first cost maintenance, renewals, etc. Perhaps the selection of a cheaper construction type even, in this case, may be false economy, but it is apparent that if there are no additional funds available and no legal machinery provided for the borrowing of the same, the garment must, to a certain extent, be cut in accordance with the cloth. This is a condition existent in many localities and one that renders it necessary in many instances, to adopt a temporary type of construction, even though the capitalized value of maintenance and renewal costs render such type of construction the more expensive in the long run. Where the location is on a new alignment this procedure may receive additional justification on the theory advanced by the railroads of placing traffic over the line at the least possible first cost and in the shortest possible time, later on building up construction standards as traffic demands increase, through a systematic annual betterment program.

Having disposed of the general features controlling type selection for highway bridges, the question of economic analysis may now be considered. This question is taken up in the next chapter.

CHAPTER III

FUNDAMENTALS OF ECONOMIC ANALYSIS

Article 1.—Source of Funds for Highway Bridge Improvements

Funds for the construction of highway bridges are in general derived from two principal sources as follows:

(a) From direct revenue (property tax revenue, revenue from auto license fees, revenue from taxes on gasoline, distillate or other fuel, etc.)

(b) By borrowing (issuance of bonds).

In either case a fund is created termed the "Capital Account" out of which is drawn the money necessary for construction.

Article 2.—Capital Costs

Let us consider first the case of a bridge costing C dollars, built of absolutely permanent construction and upon which there need be no maintenance expenditure. The capital account is charged with C dollars withdrawn but credited with a bridge worth C dollars, so that the balance remains unchanged. In other words, the total wealth of the state is in no way changed by simply exchanging C dollars in money for C dollars in bridge construction. However, the state has not by any means received a bridge free, owing to the fact that the C dollars in the form of liquid funds had an earning capacity of rC dollars per annum ($r =$ the annual interest rate) while the C dollars in bridge form has no direct earning capacity. The net result to the state, therefore, is the gain of a bridge but a loss per annum of rC dollars interest money. The *capital cost* of any bridge structure, therefore, can be represented by an annual charge representing the interest on the amount expended. This hereafter will be designated by the term rC.

Therefore, for permanent construction without maintenance and costing C dollars, the

Annual expense $=$ .... $\$rC$

Article 3.—Maintenance and Renewal Costs

Now it is apparent that no bridge can be built which will satisfy the above requirements. Regardle s of its excellence, some money for maintenance is required and at some future period the structure is bound to

wear out or become obsolete as regards type or traffic capacity and to need complete renewal.

Assume that the average annual maintenance cost is estimated at M dollars and that the probable life of the structure may be estimated at n years. The amount of money R, which must be deposited *at the end of each year* to accumulate with compound interest at $r\%$ per annum, an amount equal to C dollars in n years time is given by the expression

$$R = \frac{Cr}{[\,(1+r)^n - 1\,]} \quad \text{.....................(1)}$$

The term R obviously represents and measures the *renewal charge* against the structure. The total annual cost for capital, maintenance and renewal is therefore represented by the expression

$$E = rC + M + R \quad \text{.................................(2)}$$

Gillette's Handbook of Cost Data and other handbooks contain tables which aid in the calculation of annual renewal funds, giving the amount accumulated when one dollar is deposited annually in a fund drawing compound interest at rates from three to ten per cent and for time periods from one to fifty years. For reference call this Table I.

They also give a table which is simply the reciprocal of the one mentioned above. This table gives the amount necessary to deposit in an annual sinking or renewal fund in order to accumulate $1.00 at the end of the given time period. For reference call this Table II.

To illustrate the use of these tables let it be required to determine what annual renewal at 5% compound interest will accumulate a deposit sufficient to replace a timber trestle structure costing $24,000.00 in 15 years' time.

From Table I an annual deposit of one dollar at 5% compound interest will accumulate the sum of $21.58 in 15 years, ∴ R (the renewal fund required) = $24,000÷$21.58 = $1,112.14.

In Equation (2) provision has been made for the annual cost of (1) Capital, (2) Maintenance, (3) Renewals. In addition to these, there are several other minor items, as discussed in Article 4, which are sometimes taken into consideration.

Article 4.—Insurance Costs

A.—**Fire Insurance.** This charge is a proper one only in the case of timber construction. The state or municipality in cases of this kind does not, in general, carry commercial fire insurance, but may rather be said to carry its own insurance. A proper annual charge in a case of this kind would be an annual premium charge comparable with commercial premiums for fire hazards of similar character. Based upon ordinary

commercial fire insurance costs the following insurance rates are probably reasonable:

Untreated timber construction in isolated localities where fire control is poor, or so located as to be exposed to direct action from prairie fires, forest fires or sparks from passing trains	0.5%
Untreated timber construction protected by water barrels or under careful supervision from patrolmen or others	0.3%
Untreated timber construction protected by water supply pressure lines or city fire protection and under constant supervision (such for example as timber approaches to movable spans)	0.2%
For treated timber construction the above rates should probably be increased about	one-third

These figures are, of course, nothing but rough averages but may be used without further refinement since the resulting charge is always small.

B.—**Flood Insurance.** In certain cases the danger from flood loss is so great as to warrant the consideration of an annual item for flood insurance. In general this item is not considered except when it becomes necessary to determine the relative economy of two types of construction which differ in regard to their respective liabilities for flood loss.

In such a case the proper annual charge for flood insurance can be determined from the probable frequency of flood losses in connection with the probable service life of the structure by a consideration of the theory of probabilities as follows:

Consider a structure whose first cost is $\$C$ constructed over a waterway where the likelihood of flood destruction is once in every m years. Assume that the probable service life of the structure if no floods occur is n years.

At the time of the flood loss consider the structure to have been built for t years, then the total loss is not $\$C$ but

$$\$C\frac{n-t}{n}$$

since t years of service have already been given. The greatest possible loss would occur if the flood loss should take place immediately subsequent to a renewal and would amount to $\$C$. The minimum loss would occur when the flood loss took place just before a renewal of the structure and would amount to zero (unless, of course, the old structure should be considered as having some salvage value).

Over a period of years, therefore, it may be assumed that the average loss to be expected will amount to $\$\tfrac{1}{2}C$.

Now it is apparent that the probability of this loss occurring during

the service life n of any particular renewal will be represented by the following expression:

Probability of flood occurrence during life of this particular bridge $= \frac{n}{m}$

Therefore, over a period of years the total probable loss will be represented by the expression

$$\$\frac{nC}{2m}$$

The rate applicable for flood insurance, therefore, may be roughly taken as equal to the annual deposit at the assumed rate of compound interest which will redeem, over a period of n years time, an amount equal to

$$\tfrac{1}{2}\frac{n}{m}C$$

dollars.

The above discussion applies not only to flood losses, but also with equal force to insurance against any contingency which may happen once in m years time.

Another method of arriving at approximately the same result is to compute the probability of flood loss independent of the service life of the structure. This amount will be represented by the following expression:

Probable loss during complete flood interval =

$$\$\tfrac{1}{2}C\frac{m}{m} = \$\tfrac{1}{2}C$$

The proper insurance rate applicable in this case would be the annual deposit necessary to accumulate, over a period of m years time, a fund equal to $\frac{1}{2}C$ dollars. This method results in a lower rate than the first method when the service life of the structure is less than the probable flood interval and a higher rate when the service life of the structure is greater than the probable flood interval. This latter method is more properly applied to a calculation of insurance rates for a large group of structures while the first method is more properly applicable to a cost comparison of different types for the same opening.

If it can be reasonably assumed that the damage due to flood occurrence will result in a loss of less than 100% of the structure, or, in other words, that the bridge after the flood occurrence will have a certain salvage value S, then the term $(\frac{1}{2}C - S)$ may be substituted for $\frac{1}{2}C$ in the

above formulas. In discussion which follows later (equation 3) I represents the annual insurance cost.

Article 5.—Operation Costs

In addition to the foregoing the last item of annual expense to be considered is that of operation. Operation costs may be divided into two main classes as follows:

(a) Operation of the bridge (Designated hereafter by the term O').

(b) Traffic operation (Designated hereafter by the term O).

The first operating cost mentioned above is simply an annual charge occurring in the case of movable bridges or in connection with the operation of crossing gates, the employment of watchmen, etc.

The second operating cost is the cost to the traffic operating over the bridge. Before taking up a discussion of this phase of the subject it may be well to state certain general principles in reference thereto. In order to appreciate the justice of this traffic operation charge, it becomes necessary to consider a bridge structure as a *productive machine*. The product of this machine is principally of one kind, viz.: traffic service, and this product is measured in terms of the number of traffic units handled. There are however other by-products of highway development as a whole a certain portion of which should be credited to the bridge structures included thereon. These by-products may be summarized as follows:

(1) Development of scenic resources of state or municipality.
(2) Enhancement of abutting property values.
(3) Advertising to community and to state.
(4) Other attendant gains to community.

These are all tangible, real products and have a certain definite value. It may seem at first glance that the advertisement of the community has no real value. This, however, is far from true. Community advertisement attracts capital to undeveloped localities which is a distinct service both to capital and to the locality; moreover, a distinct service to the nation in that it aids in the development of hitherto undeveloped resources and thus adds directly to the nation's available wealth.

These, then, are the commodities which the bridge machine turns out and are marked up on the credit side of the ledger. As against these items the cost of the machine is the sum total of the interest renewal, maintenance, insurance and bridge operation costs as above outlined plus the cost of traffic operation itself. This last cost can be determined from the length of the bridge, the traffic density and the unit traffic cost, which latter value may be expressed in terms of either vehicle mile or ton-mile units. The latter is the more accurate but the former lends itself more

readily to this type of economic analysis since the use of the ton-mile requires that the traffic units be segregated. The vehicle mile unit will, therefore, be used throughout the balance of this discussion although the ton-mile unit may be employed if desired.

As an illustration of the method of arriving at traffic operation costs consider a bridge 1,000 feet in length carrying an average traffic of 1,000 vehicles per day. At an average cost of eight cents per vehicle mile the total annual operation cost amounts to

$$\left(\frac{1000}{5280}\right)(\$0.08)\ (1000)\ (365) = \$5530.30$$

It is true that the cost of traffic operation is paid for out of a different fund than that from which other annual costs are paid, it is, however, none the less a legitimate charge against the structure and should be considered in any economic comparison.

The cost per vehicle mile varies with a large number of factors and the data at hand are not sufficient to place anything more than a very rough estimate of evaluation upon the same. The following discussion of transportation costs, as affected by bridge design, may, however, prove enlightening, in so far as general principles are concerned, even though no definite or accurate quantitative conclusions are reached.

Among the factors involved in bridge construction which affect transportation costs may be mentioned the following:

(a) Character of roadway surface.
(b) Width of roadway.
(c) Horizontal alignment of structure.
(d) Grade line treatment.

Character of the Roadway Surface.—Taking up these points in order, the character of the roadway surface adopted for the bridge affects transportation costs in a variety of ways. First of all, a rough roadway surface increases the rolling resistance of the vehicle and in this measure, operates to increase the fuel consumption. The presence of a rough surface also increases the tire expense and in addition, introduces certain impact and vibration strains in the vehicle itself, which result in a more rapid depreciation of the equipment. These vibration strains are not altogether a function of the bridge floor surface, but may result from a vibration set up throughout the entire superstructure during the passage of a load. Light, pin connected steel spans, high and insufficiently braced trestle structures and other like construction will cause a general vibration of the rolling stock crossing the structure to the extent of greatly increasing the rate of depreciation on the same. In addition to the above costs which are a function of the mileage traveled by the vehicle, there

are other operating costs which are *time functions rather than mileage functions* (for example, fixed rental charges, wages for drivers or operators, garage rental, license fees, etc.) Any bridge, therefore, which operates to lower the safe rate of speed introduces an element of added transportation cost, as regards the above time functions.

Narrow Roadways.—The effect of an unduly restricted roadway is a slackening in the average speed of traffic movement and a consequent increase in time element transportation costs. Narrow roadways also operate to introduce greater liability of accident, not only major accidents, (collisions, etc.) but also injury to fenders, bumpers and other like minor damages growing out of the closely restricted clearance between vehicle and rail.

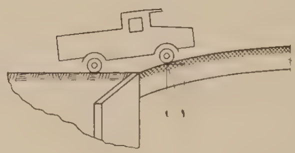


Figure 1
Impact stresses resulting from improper grade line treatment

Horizontal Alignment.—The effect of sharp curvature is to increase the liability of collision, to slacken the average safe traffic speed and to introduce an element of added wear on tires due to the longitudinal and lateral slippage which results. A general stress upon the entire vehicular mechanism also results from travel over an alignment having undue curvature.

Grade Line Treatment.—The introduction of grades operates to increase the rolling resistance of the vehicle and therefore, the fuel consumption. Sharp changes in the direction of grade, unless modified by

long vertical curves, introduce an added impact, both to the structure and to the vehicle. Conditions of this kind are generally observed either at grade vertices or at the end of the structure where the same joins with the roadway approach. (See Figure 1.)

In general it may be stated that any condition which will impair the riding qualities of the bridge roadway surface, operates to increase transportation costs. It is difficult, with the data at hand, to place any quantitative value upon the various factors hereinabove outlined and it is sincerely hoped that in the near future data suitable for a more correct evaluation of these factors may be obtained through experimental observation, thus rendering it possible to establish a more logical basis for the selection of bridge roadway types.

In the absence of further experimental data, the following table of recommended assumption as regards transportation costs per vehicle mile for various conditions of roadway surface may be used for the purpose of economic type comparison:

TYPE OF BRIDGE ROADWAY	AVERAGE TRANSPORTATION COSTS (Per vehicle mile) CLASS I	CLASS II
Loosely nailed and rough plank decks	$0.120	$0.145
Laminated timber decks (smooth)	0.102	0.120
Bituminous surfacing on timber	0.095	0.110
Permanent hard surfaced decks (average condition)	0.090	....
Permanent hard surfaced decks (smoothest condition)	0.080	....

In the above table the column headed "Class I" may be taken to represent construction wherein roadway widths are adequate and the structural design throughout is such as to produce maximum rigidity or freedom from vibration under loads. "Class II" construction may be taken to represent structures having a narrowly restricted roadway or structures which are vibratory under traffic or which, for any other reason, introduce an added impediment to transportation.

The values given in the above table are also on the basis of level tangents. For various conditions of curvature and grade line treatment the cost of operation is undoubtedly increased. Unfortunately, data are not at hand whereby a definite value can be placed upon these two factors. In the absence of further data it may be assumed for the purposes of type comparison, as herein outlined, that the unit costs in the above table are increased 3% for each percent of grade above 2% and are increased 1% for each degree of curvature above 10°.

The expense side of the ledger may now be totaled up as follows:

$$\text{Total Annual Cost} = rC + M + R + I + O + O' \quad \text{..........(3)}$$

Article 6.—Rental Values

The earning capacity of the bridge has already been stated as follows:

(1) Traffic service.
(2) Development of scenic resources.
(3) Enhancement of abutting property values.
(4) Advertising to the community and to the state.
(5) Other attendant gains.

The last four items are rather difficult of exact evaluation. For example, it is difficult to determine the extent to which a beautiful concrete arch structure operates to increase property values adjacent thereto, likewise the difference between this type of construction and a timber bridge in this regard. It is difficult to place an exact value on the community advertisement afforded by a fine example of bridge architecture. It is definitely known, however, that such value does exist and that in each case the earning value bears a roughly fixed ratio to the total first cost. In view of this fact, the suggestion has been made by some who have given considerable thought to this question that the benefits mentioned hereinabove and listed under items 2, 3, 4 and 5 be taken care of by the introduction of a fixed rental charge of p percent of the first cost, deducting this value from the annual cost and prorating the residue in accordance with the number of traffic units served. The proper rental value to be applied in any given case depends upon the weight which is given to the above factors. For primary highways improved to first-class standards especially where attempt is made at scenic beautification it appears that the rental value of the structure may be placed as high as two percent. For secondary roads a rental value of from one-half percent to one percent should probably be employed. The reason for this difference in rental percentage lies in the fact that such features as "development of scenic resource" and "advertising to community" increase in importance with the traffic density and general importance of the highway.

Article 7.—Final Economic Equations

Having developed the underlying principles, the general economic equation may now be written as follows:

Case I, All Conditions Considered

$$\text{Total Annual Cost} = rC + M + R + I + O + O' - pC \quad \text{(4)}$$

Case II, Neglecting Insurance Costs

$$\text{Total Annual Cost} = rC + M + R + O + O' - pC \quad \text{(5)}$$

Case III, Neglecting Operating and Rental Charges

$$\text{Total Annual Cost} = rC + M + R \quad \text{(6)}$$

Case IV, Consideration of Salvage Values

It may be that at the end of its economic service period a bridge structure may possess a certain salvage value. For example, perhaps part of the abutments or piers may be used (with modification, if necessary) in the new construction. If the use of any portion of the present structure operates to effect a saving of S dollars in the total cost of a future replacement then the term S may be said to represent the salvage value of the structure at the end of its service life. In this case the term R, representing the renewal cost in the above equations, should be multiplied by the factor $\left[\frac{C-S}{C}\right]$ since the sinking fund need only to accumulate the additional cost ($C - S$) of a replacement. For example, the last equation hereinabove would now read:

$$\text{Total Annual Cost} = rC + M + R\frac{(C-S)}{C} \quad \dots\dots\dots\dots\dots\dots\dots (7)$$

Data have already been given whereby an estimate can be made regarding the values of O and p. The value of O', the annual operation cost of the bridge itself (movable bridge operation, watchmen, etc.) is a special cost and is generally known (within reasonably close limits) in advance. The value of I (annual insurance cost) may also be determined, once the value of C is known from considerations which have gone before. It, therefore, remains to determine the proper estimated values to use for the following:

C—the estimated first costs for different construction types.

M—the probable average annual maintenance costs for various construction types.

R—the annual renewal costs for various construction types.

It is the purpose of the chapters which follow to submit data for the ordinary highway bridge types encountered in practice, which will enable the engineer to solve the above economic equations for each proposed type and thus, in connection with the considerations of hydraulic demand, traffic service, etc., as set forth in Chapter II, to properly select the type of construction best suited to the individual needs of the site.

CHAPTER IV
BRIDGE TYPES

GENERAL DISCUSSION AND FIRST COST DATA

Article 1.—Introductory

The purpose of this chapter is to submit data from which comparative first cost estimates for various construction types may be prepared, thus evaluating the term C of the equations given in Chapter III, and also to discuss under each type the question of its peculiar advantages, and disadvantages and its general scope and field of utility.

In general only the more commonly used types of construction have been included for the reason that special construction types are generally designed for special conditions and can be intelligently discussed only when those particular conditions are known. The cost data curves and type illustrations are taken in nearly every case from the writer's own practice and therefore do not exactly fit any other set of standards. This construction represents a design practice that is doubtless about as conservative as regards details, loading and stress requirements as any highway bridge practice in the United States so that the quantities are probably as large as those for any standard state construction and somewhat larger than is in use in some of the states. The difference in standard highway bridge practice over the country is becoming less and less each year and the general tendency is toward the specification requirements which have been adopted by the American Association of State Highway Officials. These specifications are nearly identical with those for which the cost data curves hereinafter given were prepared. For this reason it is felt that the quantity curves hereinafter included may furnish a basis for the preparation of preliminary estimates for the comparison of different types exact enough for use anywhere in the country.

In order that the curves may be more readily interpreted the following digest of the specification requirements adopted by the American Association of State Highway Officials is given:

Digest of Load Specification Requirements

Adopted by the
American Association of State Highway Officials

1. DEAD LOAD

The dead load shall consist of the weight of the structure complete, including the weight of the roadway floor, car tracks, pipes, conduits, cables, or other public utility services supported thereby.

In the case of structures having concrete slab floors, an adequate allowance shall be made in the design dead load to provide for the weight of a suitable wearing surface. This allowance will depend upon the type of wearing surface contemplated; it shall be in addition to the weight of any monolithically placed concrete wearing surface; and shall be not less than 15 pounds per square foot of roadway.

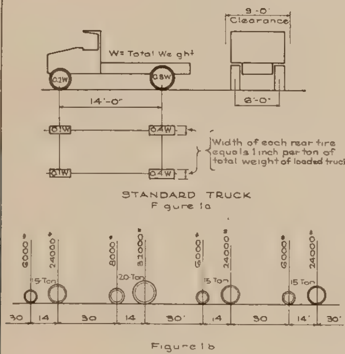


Figure 1
STANDARD TRUCK LOADING
ADOPTED BY THE AASHO

2. LIVE LOAD

Highway Loads for Bridge Floors. The integral parts of bridge floor systems, including their direct connections to trusses, girders and viaduct towers and bents, shall be designed for the maximum stresses produced by truck concentrations. Floor beam hangers and all integral members or parts of trusses and girders subjected to the direct action of floor loads and impacts shall correspondingly be designed for truck concentrations. The truck dimensions and weight distributions used for design purposes shall be those of the typical or standard trucks shown in Figure 1. These standard trucks are designated by the letter **H**, followed by a numeral indicating for each class its gross or total weight in tons.

The class of loading used shall be one of the following:

Loading H20: 20-ton trucks.
Loading H15: 15-ton trucks.
Loading H12½: 12½-ton trucks or an alternate load of one 15-ton truck.
Loading H10: 10-ton trucks.

Highway Loads for Trusses and Girders. The trusses and girders of bridge spans and the columns of viaduct towers and bents shall be designed for the

stresses produced by a load on each traffic lane composed of a uniform load per linear foot of lane with a concentrated load so located longitudinally therein as to produce maximum stresses. The concentrated load shall be considered as uniformly distributed transversely on a line having a length equal to the width of the lane. The standard truck clearance width of 9 feet shall be assumed as constituting the width of one traffic lane.

The class of loading used shall be one of the following:

Loading H20: A total load on each traffic lane composed of a uniform load of 600 lb. per lin. ft. and a single concentrated load of 28,000 lb.

Loading H15: A total load on each traffic lane composed of a uniform load of 450 lb. per lin. ft. and a single concentrated load of 21,000 lb.

Loading H12½: A total load on each traffic lane composed of a uniform load of 375 lb. per lin. ft. and a single concentrated load of 17,500 lb.

Loading H10: A total load on each traffic lane composed of a uniform load of 300 lb. per lin. ft. and a single concentrated load of 14,000 lb.

(Note: Loading H15 is 75%, Loading H12½ is 62½% and Loading H10 is 50% of Loading H20.)

Loading H20 is approximately equivalent to the typical truck loading shown in Figure 1. This loading consists of one 20-ton truck followed by, or preceded by, or both followed and preceded by a line of 15-ton trucks of indefinite length and assumed to occupy a clearance or lane width of 9 feet.

3. LOAD CLASSIFICATION OF BRIDGES

Bridges shall be classified or rated in relation to their capacities for safely supporting highway loads or a combination of highway and electric railway loads. In general, the division into classes and the corresponding loadings shall be as follows:

Class AA: Bridges supporting specially heavy highway traffic units in locations where the passage of such loads is frequent. Class AA bridges shall be designed for Loading H20.

Class A: Bridges supporting normally heavy highway traffic units with occasional specially heavy loads. Class A bridges shall be designed for Loading H15.

Class B: Bridges supporting normally light highway traffic units with occasional heavier loads. Class B bridges shall be designed for Loading H12½.

Class C: Bridges of a temporary or semi-temporary nature, supporting light highway traffic units. Class C bridges shall be designed for Loading H10.

Class D: Bridges supporting electric railway traffic in addition to highway traffic. The highway loads may correspond to any one of the four classes above specified. The electric railway loads shall be as specified.

Application of Loads to Girders and Trusses. Trusses and girders shall be designed to support as many traffic lanes as the width of roadway will permit, assuming them to be placed symmetrically with regard to the roadway center line.

To provide for an increase in truss and girder stresses resulting from the passage of eccentrically placed loads and for a decrease in traffic lane intensity

for increasing widths of roadway, the stresses obtained by the application of the above loading shall be multiplied by the coefficient "C" given by the following formulas:

Case I. When "W" is less than 18 feet,

$$C=\frac{W}{9}$$

Case II. When "W" is equal to or greater than 18 feet,

$$C=\frac{18+W}{18n}$$

Where,

W = the width of roadway for bridges with two main girders or trusses; or the distance center to center of girders or trusses for bridges with more than two main girders or trusses.

n = number of lanes of traffic.

Application of Loads to Floor System. Bridge floor systems shall be designed to support as many trucks, not exceeding four, as the width of roadway will permit. For bridges involving the use of Loading H12½, the alternate single truck specified shall be used whenever maximum stresses are produced thereby.

When the design of the floor system involves the placing of trucks adjacent to curbs, the extreme position of a truck shall be assumed as that in which the center of the outside wheel is 1 ft. 6 in. from the inside edge of the curb.

In the design of floorbeams and their supports the following percentages of the resultant live load stresses shall be used:

One or two trucks	100 per cent
Three trucks	90 per cent
Four trucks	80 per cent

4. IMPACT

All live load stresses, except those due to sidewalk loads and centrifugal, tractive and wind forces, shall be increased by an allowance for dynamic, vibratory and impact effects.

For end floorbeams, floorbeam hangers, columns supporting floorbeam concentrations and all floorbeam connections, the impact allowance shall be 60% of the live load stress.

For all other portions of structures, the impact allowance or increment is expressed as a coefficient of the live load stress varying with the loaded length of the structure and the width of the roadway area. Its intensity is determined by the following formulas in which

I = impact coefficient.

L = loaded length in feet, producing the maximum static stress in the member considered.

W = the width of roadway for bridges with two main girders or trusses; or the distance center to center of girders or trusses for bridges with more than two main girders or trusses.

When "W" is equal to or less than 18 feet,

$$I = \frac{L + 250}{10L + 500}$$

When "W" is greater than 18 feet,

$$I = \left(\frac{36}{W+18}\right)\frac{(L + 250)}{(10L + 500)}$$

For highway loads, the maximum value of "I," as given by the above formulas, shall not exceed 0.30.

The following standard construction types are included in the discussion hereinafter given:

Superstructures

Short Span Superstructures.

1. Timber trestle superstructures.
2. Steel I-beam spans with timber floor.
3. Steel I-beam spans with concrete deck and railing.
4. Concrete incased I-beam spans.
5. Reinforced concrete slab spans.
6. Reinforced concrete multiple beam spans.
7. Reinforced concrete deck girder spans.
8. Reinforced concrete through girder spans.

Longer Span Superstructures.

1. Timber trusses uncovered (pony spans and A-frames).
2. Timber trusses uncovered (through high truss spans).
3. Housed timber through truss spans.
4. Steel pony truss spans, timber floor.
5. Steel pony truss spans, concrete floor.
6. Steel through truss spans, timber floor.
7. Steel through truss spans, concrete floor.
8. Timber deck truss spans.
9. Steel deck truss spans, timber floor.
10. Steel deck truss spans, concrete floor.
11. Steel deck plate girder spans.
12. Steel through plate girdle spans.
13. Reinforced concrete arch spans.

Substructures

1. Pile bents.
2. Frame bents on concrete pedestals.
3. Frame bents on mud sills.
4. Mass concrete pedestal piers and abutments.
5. Steel viaduct towers.
6. Reinforced concrete viaduct construction.
7. Heavy gravity abutments (masonry).
8. Heavy gravity abutments (concrete).
9. Concrete piers, "Dumb Bell" type.
10. Concrete piers, other types.
11. Reinforced concrete abutments and wing walls.

Miscellaneous Construction

1. Filled approaches.
2. Concrete box culverts.
3. Concrete or masonry arch culverts.
4. Concrete circular culverts.
5. Concrete pipe culverts.
6. Metal pipe culverts.
7. Other culvert types.
8. Retaining walls.

In general, cost and quantity curves will be given only for Class A and Class B structures.

SECTION I.—SHORT SPAN SUPERSTRUCTURES

Article 2.—Timber Trestle Superstructures

This type of structure is suitable for locations when short span construction is permissible. In connection with timber bents, its use is restricted to such locations as are free from danger from drift or ice, and for bridging dry canyons and ravines. Construction of this type is often employed for approaches to larger spans, the trestle type extending over the overflow flat or region of comparatively slack and shallow water. Its principal advantage is low first cost; its principal disadvantage is high maintenance, short service life and fire hazard.

Figures 2 and 3 are sketches showing two types of trestle superstructure of the type above described. Figure 2 is designed for Class A loading and has a 6″ laminated or "strip" type deck. Figure 3 represents a Class B loading design with a lighter type of deck, both types are designed with heavy timber wheel guard and hand rail, bolted throughout and fitted with every detail necessary for a maximum service life. These represent one of the highest types of trestle superstructure and a type which has a greater first cost than many in common use heretofore. Figure 4 is a photograph of a trestle of this type.

The materials needed for the trestle superstructure types above illustrated are tabulated in Figures 5, 6 and 7. These quantities include every item except superintendence and overhead as noted on the figures. The same data in curve form and grouped a little differently are given in Figures 8, 9 and 10, while Figures 11, 12 and 13 are *cost* curves based on various assumed unit costs per M. F. B. M. for the timber erected and painted. These curves and tables are doubtless self explanatory except perhaps the item labelled "turnouts." This designation refers to the large chamfered entrance posts and the flared extension of handrail and wheel guard (See Figures 2, 3 and 4). These are kept separate

from the other quantities as there will be but four of these entrance turn-outs regardless of the length of trestle.

It will be observed that the labor items in the above curves and

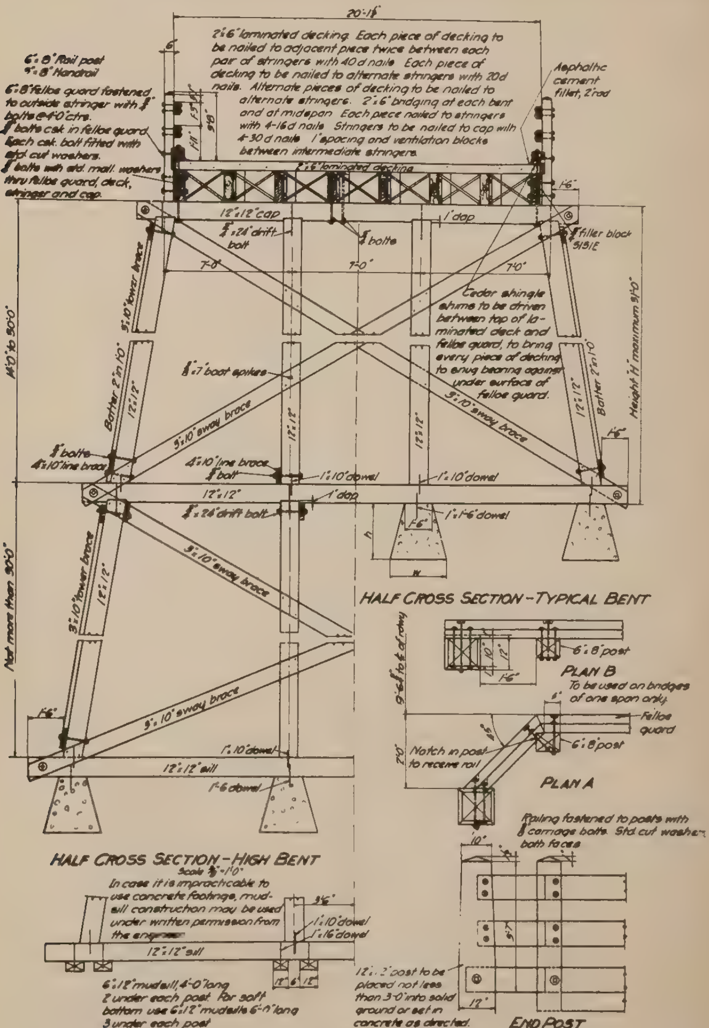


Figure 2. Cross section, Class A trestle superstructure, frame bent substructure.

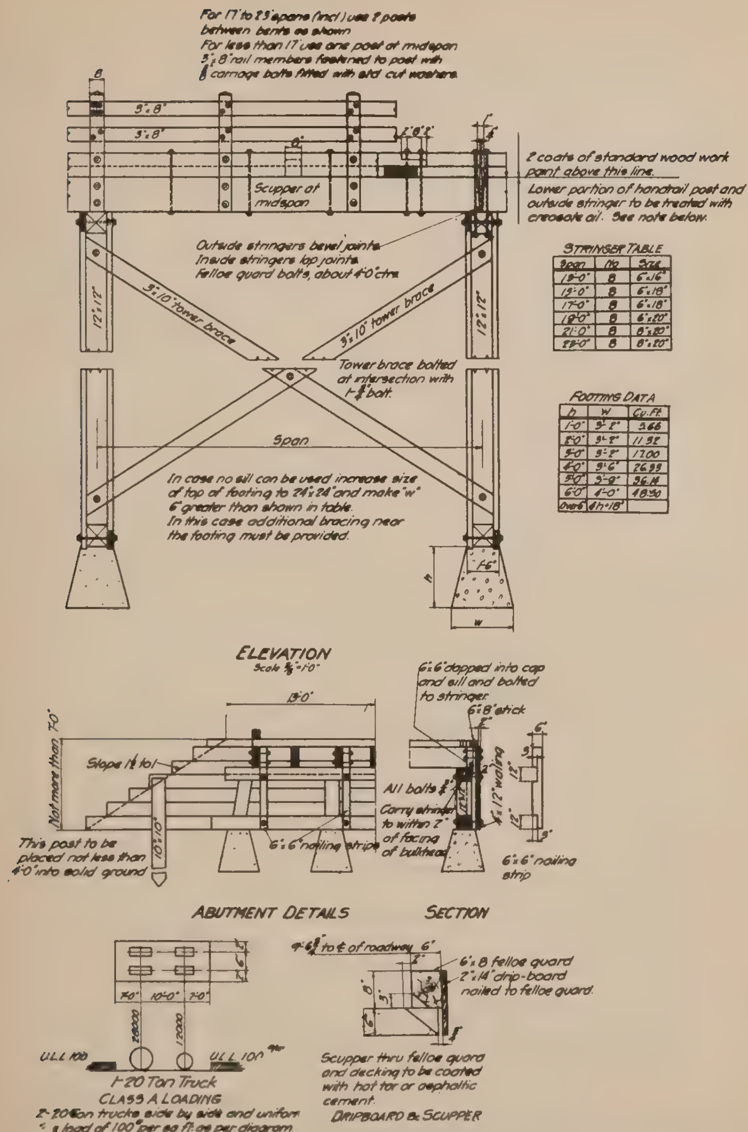


Figure 2a. Elevation, Class A trestle superstructure, frame bent substructure.

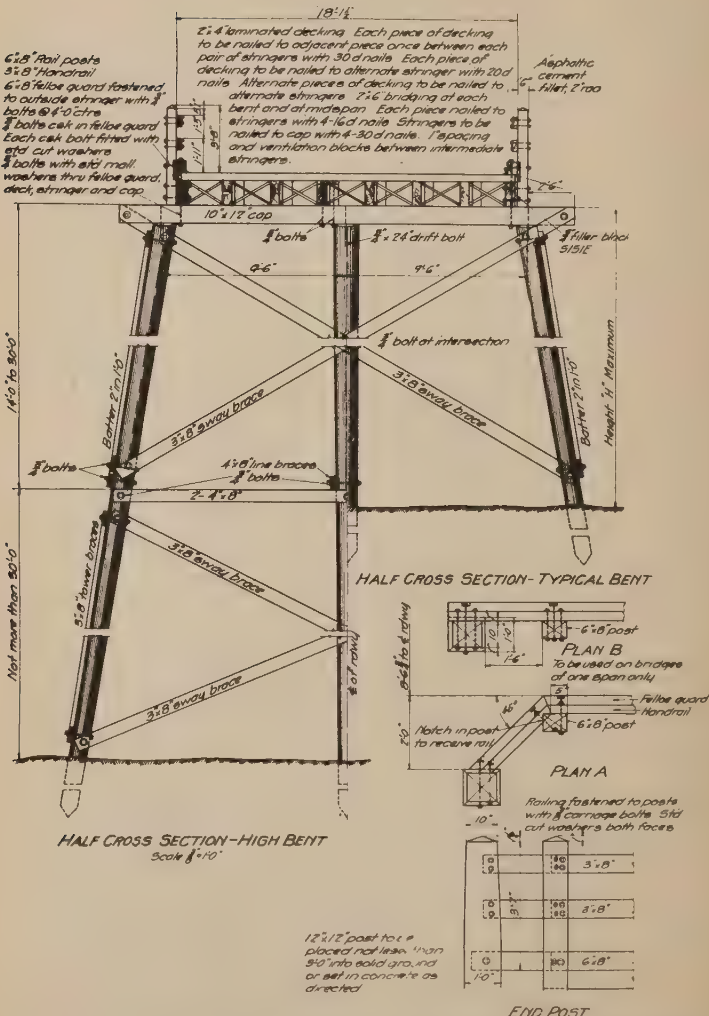


Figure 3. Cross section, Class B trestle superstructure, piling substructure.

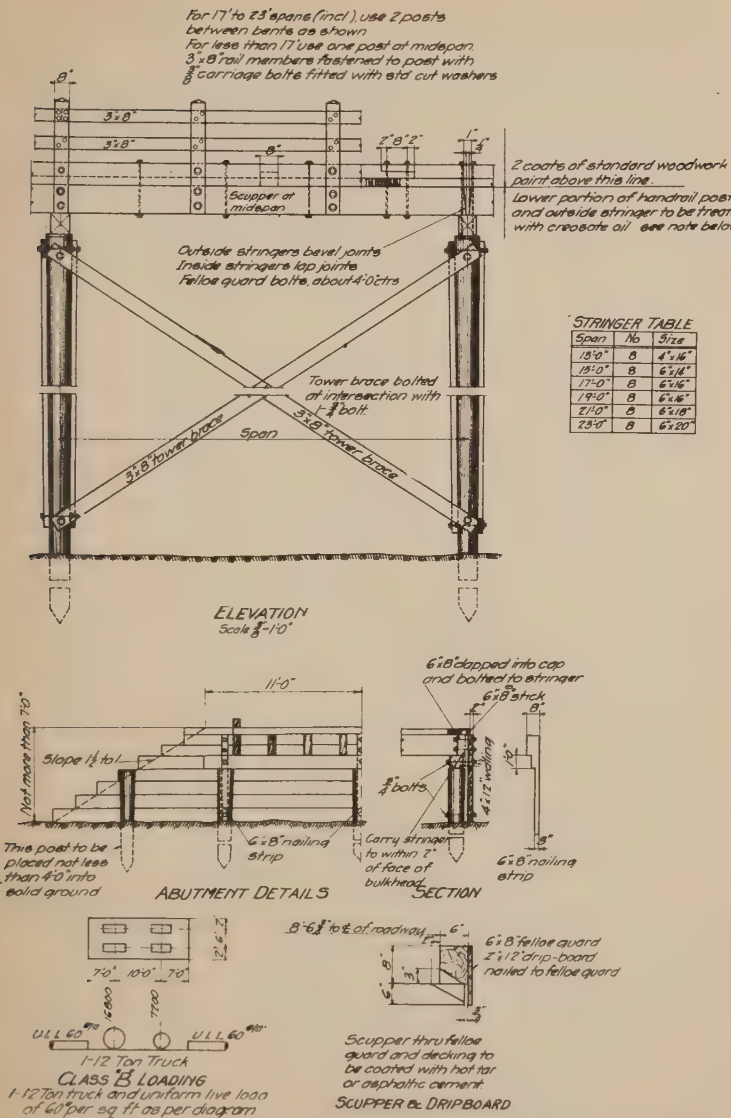


Figure 3a. Elevation, Class B trestle superstructure, piling substructure.

Figure 4. Typical low frame trestle construction.

tables are given in terms of "man-hours," thus rendering the table of value regardless of current wage scales. The quantity curves may be converted into cost data by applying prevailing unit prices for labor and material and adding the cost of superintendence and overhead.

As above stated, these data represent costs for a very high class of trestle construction, designed for a maximum service life and heavy traffic conditions. Temporary construction can be effected at a cost much less than that for the types shown.

It will be observed that the Class A and Class B loading require-

FIGURE 5

QUANTITIES OF MATERIAL AND LABOR FOR STANDARD TIMBER SUPERSTRUCTURES

Class "A" Loading — 20' Roadway

	ITEM	QUANTITIES PER LINEAL FOOT OF SPAN				
		Lumber FBM	Nails Lbs	Bolts Lbs	Paint & Preservative gals	Labor Man Hrs.
13' SPAN	Handrail & Wheelguard	30	008	797	010	300
	Decking	120	545			132
	Bridging & Misc	7	018			032
	Stringers	69	021	077		166
	Total	226	592	874	010	628
15' SPAN	Handrail & Wheelguard	28	008	790	010	300
	Decking	120	545			132
	Bridging & Misc	6	016			027
	Stringers	77	018	083		185
	Total	231	587	873	010	644
17' SPAN	Handrail & Wheelguard	30	008	838	010	300
	Decking	120	545			132
	Bridging & Misc	5	014			024
	Stringers	76	016	073		190
	Total	231	583	911	010	646
19' SPAN	Handrail & Wheelguard	30	008	833	010	300
	Decking	120	545			132
	Bridging & Misc	5	013			021
	Stringers	84	014	068		202
	Total	239	580	901	010	655
21' SPAN	Handrail & Wheelguard	29	008	754	010	300
	Decking	120	545			132
	Bridging & Misc	4	011			019
	Stringers	112	013	061		270
	Total	265	577	815	010	721
23' SPAN	Handrail & Wheelguard	28	008	688	010	300
	Decking	120	545			132
	Bridging & Misc	4	010			018
	Stringers	111	012	056		280
	Total	263	575	744	010	730
Additional material for two bulkheads & four turnouts		2819	424	203	–	700

Note Labor does not include superintendence or overhead

FIGURE 6

QUANTITIES OF MATERIAL AND LABOR FOR STANDARD TIMBER SUPERSTRUCTURES

Class "A" Loading — 18' Roadway

	ITEM	QUANTITIES PER LINEAL FOOT OF SPAN				
		Lumber FBM	Nails Lbs	Bolts Lbs	Paint & Preservative Gals	Labor Man hrs
13' SPAN	Handrail & Wheelguard	30	008	797	010	300
	Decking	108	470			120
	Bridging & Misc	5	017			030
	Stringers	60	018	077		145
	Total	203	513	874	010	595
15' SPAN	Handrail & Wheelguard	28	008	790	010	300
	Decking	108	470			120
	Bridging & Misc	5	015			027
	Stringers	67	016	083		162
	Total	208	509	873	010	609
17' SPAN	Handrail & Wheelguard	30	008	838	010	300
	Decking	108	470			120
	Bridging & Misc	5	013			024
	Stringers	67	013	073		167
	Total	210	504	911	010	611
19' SPAN	Handrail & Wheelguard	30	008	833	010	300
	Decking	108	470			120
	Bridging & Misc	4	012			021
	Stringers	74	011	068		177
	Total	216	501	901	010	618
21' SPAN	Handrail & Wheelguard	29	008	754	010	300
	Decking	108	470			120
	Bridging & Misc	4	011			018
	Stringers	98	011	061		236
	Total	239	500	815	010	674
23' SPAN	Handrail & Wheelguard	28	008	688	010	300
	Decking	108	470			120
	Bridging & Misc	4	010			017
	Stringers	97	010	056		245
	Total	237	498	744	010	682
Additional material for two bulkheads & four turnouts		2719	424	203	–	700

Note Labor does not include superintendence or overhead

FIGURE 7
QUANTITIES OF MATERIAL AND LABOR FOR STANDARD TIMBER SUPERSTRUCTURES
Class "B" Loading — 18' Roadway

	ITEM	Quantities per lineal foot of span: Lumber FBM	Nails Lbs	Bolts Lbs	Paint & Preservative Gals	Labor Man hrs
13' SPAN	Handrail & Wheelguard	29	007	773	010	300
	Decking	72	253			080
	Bridging & Misc.	6	018			030
	Stringers	46	021	077		110
	Total	153	299	850	010	520
15' SPAN	Handrail & Wheelguard	27	007	664	010	300
	Decking	72	253			080
	Bridging & Misc	5	016			027
	Stringers	60	018	064		144
	Total	164	294	728	010	551
17' SPAN	Handrail & Wheelguard	30	007	767	010	300
	Decking	72	253			080
	Bridging & Misc	4	014			024
	Stringers	68	016	059		165
	Total	174	290	826	010	569
19' SPAN	Handrail & Wheelguard	29	007	740	010	300
	Decking	72	253			080
	Bridging & Misc	4	013			021
	Stringers	67	014	053		161
	Total	172	287	793	010	562
21' SPAN	Handrail & Wheelguard	28	007	724	010	300
	Decking	72	253			080
	Bridging & Misc	4	011			018
	Stringers	75	013	048		181
	Total	179	284	772	010	579
23' SPAN	Handrail & Wheelguard	27	007	709	010	300
	Decking	72	253			080
	Bridging & Misc	4	010			017
	Stringers	83	012	044		202
	Total	186	282	753	010	599
Additional material for two bulkheads & four turnouts		2719	424	203	—	700

Note Labor does not include superintendence or overhead

ments given in Figures 2 and 3 and some of the other drawings which follow are somewhat heavier than the corresponding A. A. S. H. O. requirements. In view of the fact, however, that no impact has been added to these loadings for design purposes, the resulting member sizes are very nearly the same as if the A. A. S. H. O. specifications had been employed.

ARTICLE 3.—STEEL I-BEAM SPANS WITH TIMBER FLOOR

Figure 16 is a sketch showing typical construction of this type and

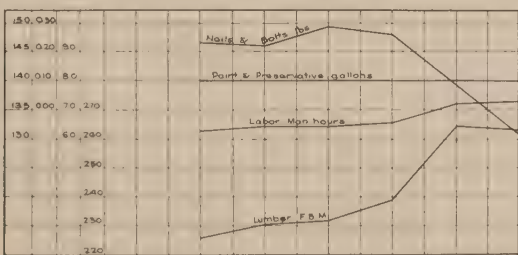


FIGURE 8
QUANTITY CURVES FOR ONE LINEAL FOOT OF STANDARD TIMBER SUPERSTRUCTURE
Class "A" Loading — 20' Roadway

Bulkheads and turnouts not included See figure 1
Note Labor item does not include superintendence or overhead

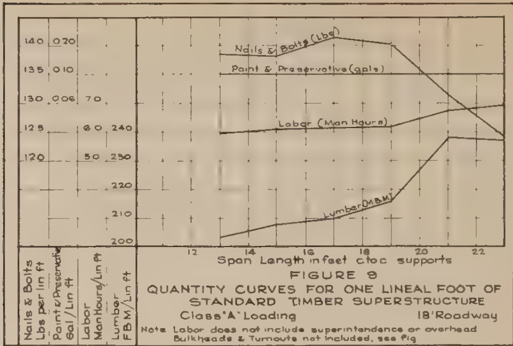


FIGURE 9
QUANTITY CURVES FOR ONE LINEAL FOOT OF STANDARD TIMBER SUPERSTRUCTURE
Class "A" Loading 18' Roadway
Note Labor does not include superintendence or overhead
Bulkheads & Turnouts not included, see Fig

Figures 14 and 15 contain quantity curves for the same.

This type of construction is adapted to the same general conditions as those which render timber trestle superstructures suitable. In this case, however, spans up to 32 feet are entirely practicable and spans on up to 40 feet are feasible although the deflection in this latter case is rather more than is desirable unless the spans be stiffened at extra expense.

The fire hazard for this type of construction is less than that for the timber, which makes the same admirably suited for short spans over railway tracks in connection with grade separation work unless a more permanent type of construction (concrete or steel plate girders) is desired.

The relative advantage of this type over the timber superstructure increases as conditions favorable to timber decay are encountered. Thus, in damp climates where timber decay is rapid I-beam spans may show to advantage while for arid regions the advantage rapidly decreases,

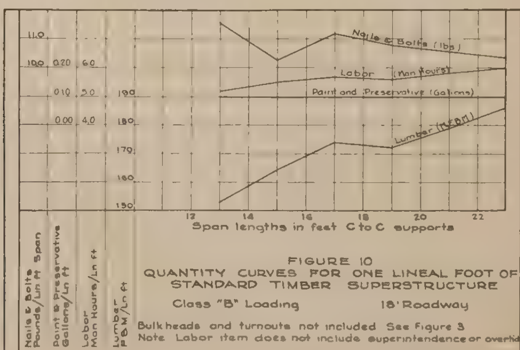


FIGURE 10
QUANTITY CURVES FOR ONE LINEAL FOOT OF STANDARD TIMBER SUPERSTRUCTURE
Class "B" Loading 18' Roadway
Bulkheads and turnouts not included See Figure 3
Note Labor item does not include superintendence or overhd

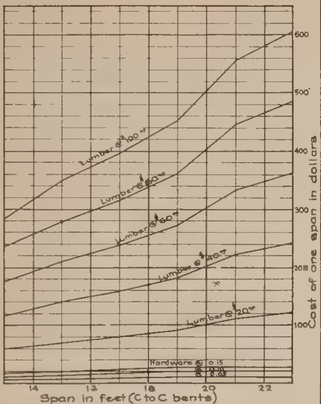


FIGURE 11
COST CURVES FOR
ONE SPAN
STANDARD TIMBER SUPERSTRUCTURE
Class "A" Loading 20' Roadway
Notes: Bulkheads & turnouts are not included above.
Price for lumber includes nails, erecting & paint
Hardware includes bolts, washers, etc.

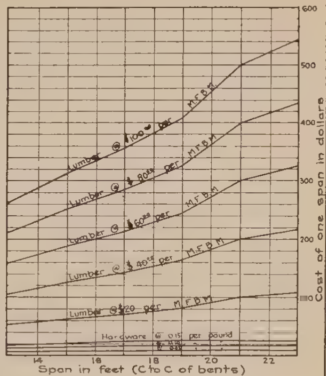


FIGURE 12
COST CURVES FOR
ONE SPAN
STANDARD TIMBER SUPERSTRUCTURE
Class "A" Loading 18' Roadway
Notes: Bulkheads & turnouts are not included above.
Price for lumber includes nails, erection & paint.
Hardware includes bolts, washers, etc.

especially if good structural timber is available locally. This type of construction is more costly than timber, but more permanent; it can be

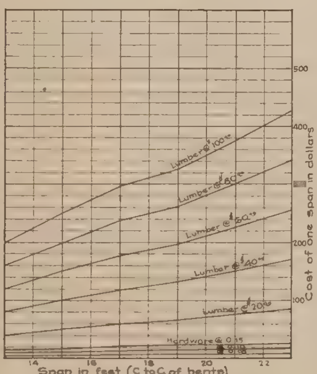


FIGURE 13
COST CURVES FOR
ONE SPAN
STANDARD TIMBER SUPERSTRUCTURE
Class "B" Loading 18' Roadway
Notes: Bulkheads & turnouts are not included above.
Price for lumber includes nails, erection & paint
Hardware includes bolts, washers, etc.

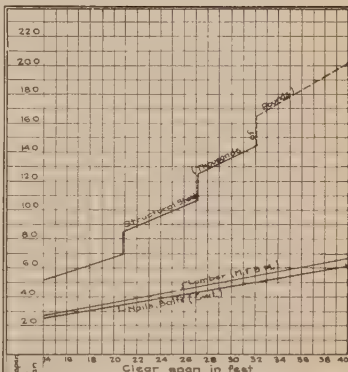


FIGURE 14
QUANTITY CURVES FOR
ONE SPAN
STANDARD "I" BEAM SUPERSTRUCTURE
Class "A" Loading. 7-"I" Beams. 20' Roadway
Timber deck & handrail
Note: Above curves do not include bulkheads or turnouts for which see figure 5

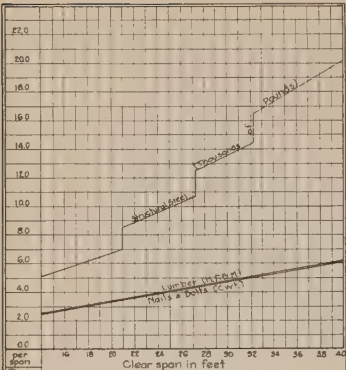


FIGURE 15

QUANTITY CURVES FOR ONE SPAN STANDARD "I" BEAM SUPERSTRUCTURE

Class "A" Loading, 7" "I" Beams, 18' Roadway
Timber deck & handrail

Note: Above curves do not include bulkheads or turnouts for which see figure 6

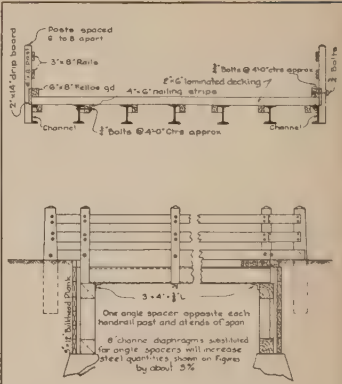
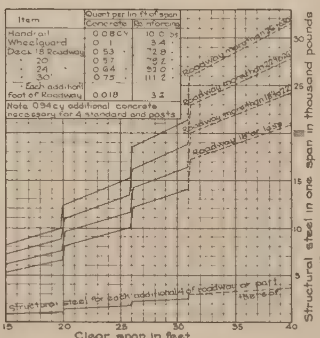


FIGURE 16

SHOWING GENERAL TYPE OF DESIGN FOR STANDARD "I" BEAM SPAN
Timber Deck and Handrail



Item	Quant per lin ft of span	
	Concrete	Reinforcing
Handrail	0 08 CY	10 0 lbs
Wheelguard	0 1	3 4
Deck 18 Roadway	0 53	72 8
20	0 57	79 2
24	0 64	92 0
30'	0 75	111 2
Each additional Foot of Roadway	0 018	3 2

Note 0 94 cy additional concrete necessary for 4 standard end posts

FIGURE 17

QUANTITY CURVES FOR ONE SPAN STANDARD "I" BEAM BRIDGE
Superstructure only
Class "A" Loading   Concrete deck and handrail

designed for a future concrete floor, thus rendering it possible to convert the span into a still more permanent type later on, if desired. The type of handrail and wheel guard employed in the design shown is the same as for the timber superstructure design illustrated in Figures 3 and 4 and its appearance in the field is much the same.

The spacer members are 3″x4″x⅜″ angles as shown. If 8″ channel diaphragms are employed as indicated on the note (Figure 16) the structural steel quantities will be increased 3%.

It frequently happens that, for long sections of trestle approach, there may be one or more individual spans which, because of bad skew crossings of canals or ditches, or crossings of one or more railway tracks necessitate a span length above the practicable limit for timber trestle construction (about 23 feet). In this case the choice becomes one between a timber "A" frame and an I-beam span with timber deck. In cases of this kind the I-beam type presents a much more sightly appearance, requires less maintenance and is to be preferred even at a slightly greater cost.

Article 4.—I-Beam Spans with Concrete Deck and Rail

This type, illustrated in Figure 18, is more permanent and much more sightly than the type with timber deck, but naturally higher in first cost. In the majority of cases there is very little to warrant the selection of this type in preference to the reinforced concrete multiple beam type or deck girder type hereinafter described. The maintenance cost is probably somewhat higher for the I-beam type. The question of selection is, therefore, entirely one of economics.

Quantity curves for this type of design are given in Figure 17 for various roadway widths and for Class A loading. The break in the curves denotes the points at which the size of beams is changed. The quantities above 32 feet are shown in dotted line to indicate that this type rarely finds a field of economic utility above this point.

The type of hand rail used with this span is indicated in Figure 18. This is the particular type of hand railing adopted by the writer and the table of quantities given at the upper left hand corner of Figure 17 are for this particular design and applicable to no other. Obviously any desirable type of hand rail may be employed in this connection.

Article 5.—Concrete Encased I-Beam Spans

This type, shown in Figure 19, is in direct cost competition with the reinforced concrete slab, beam, and girder types described hereinafter, the choice between any of these types being entirely one of economics. It is probably true that the maintenance cost over a long period

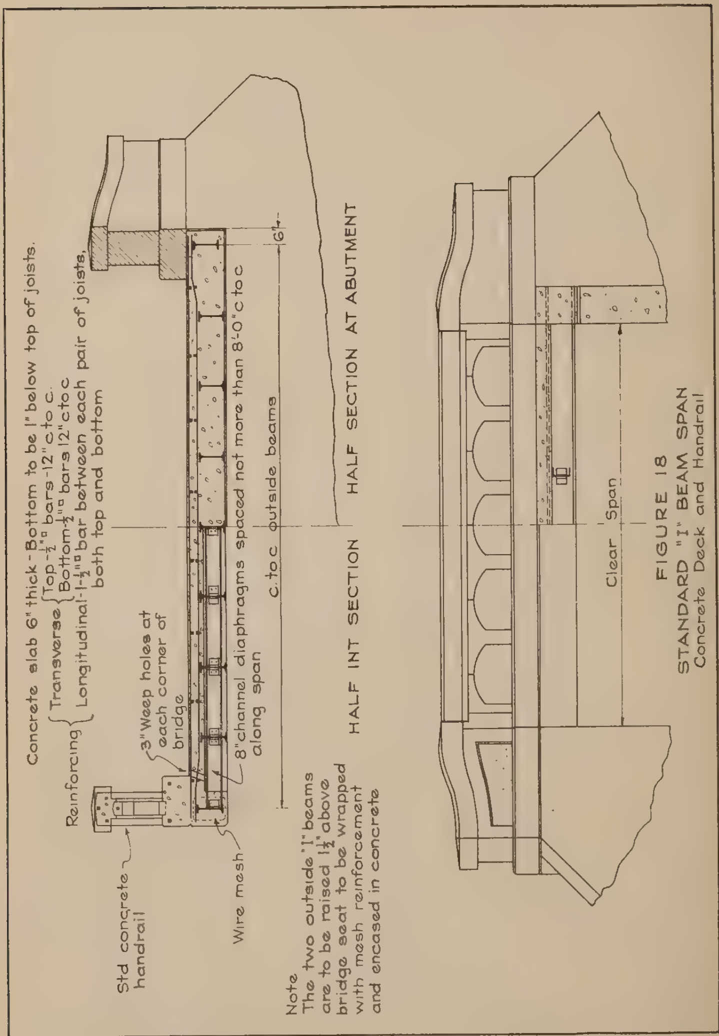


FIGURE 18
STANDARD "I" BEAM SPAN
Concrete Deck and Handrail

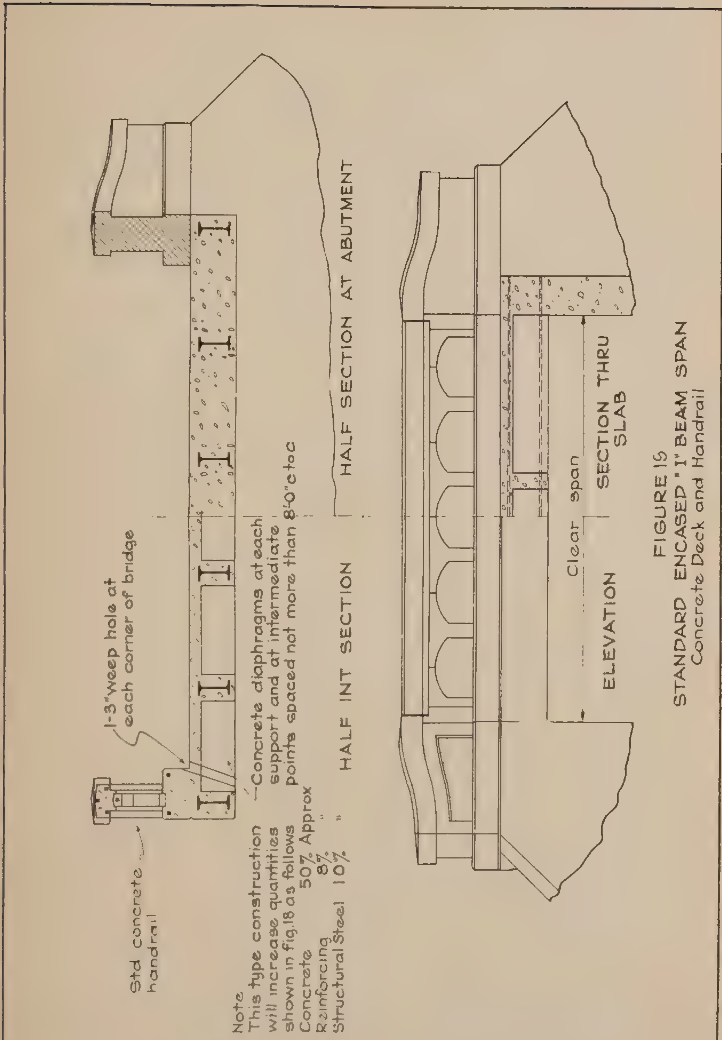


FIGURE 19
STANDARD ENCASED "I" BEAM SPAN
Concrete Deck and Handrail

of years will be greater for the encased I-beam construction than for its reinforced concrete competitor. This predication is based upon observation of several old spans of this type wherein the concrete encasement showed a marked tendency to crack and peel away from the I-beams.

Article 6.—Reinforced Concrete Slab Spans, Multiple Beam Spans, and Deck Girder Spans

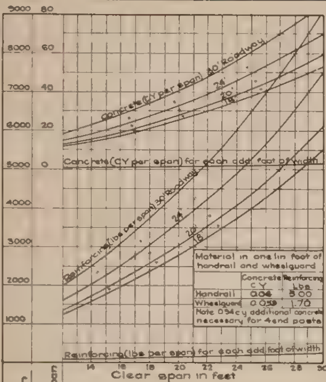
These three types are considered together because they differ only in cost, in other words, the three types are in price competition only and this simply as regards *first cost* since maintenance, renewals, etc., etc., are as far as is now known, about identical in every case. There is a slight difference in the vertical clearance requirements for the different types which may effect the selection, otherwise the choice is a matter of lowest first cost.

Figures 20, 23 and 26 are quantity curves for the above three superstructure types for different roadway widths and Class A loading while Figures 21a, 21b, etc., 24a, 24b, etc., and 27a, 27b, etc., are cost curve groups computed for varying unit prices. The deck girder quantities above 50-foot spans are approximate only. Figure 22 is a typical cross section of a reinforced concrete slab span such as furnished the basis for the preparation of the curves of Figure 20. Figure 25 is a typical design employing R. C. multiple beam construction with "buried abutments." Figure 28 illustrates this same type used for a single span crossing over a drainage canal, short wings being employed in this case. Figure 29 illustrates the use of the R. C. deck girder type for a R. R. overcrossing viaduct. These designs are given only in such detail as will illustrate the general nature of their use. Figures 30, 31 and 32 are photographs of actual construction employing these superstructure types.

It is observed that the ranges of span lengths for the three above types are not identical but cover distinct but overlapping fields. For spans up to 16 feet the slab type is doubtless to be preferred. For spans between 24 and 30 feet the multiple beam construction appears most suitable, while for spans in excess of 32 feet the deck girder type has an unrestricted field.

The choice in any case involves only the matter of clearance requirements and first cost as above stated. A graphic comparison of quantities and vertical clearance obstruction for a standard 24 foot roadway is given, for each of these types, in Figure 33. The vertical clearances will be the same for any roadway width while the relative quantities will be much the same.

It should be noted in passing that the quantity ratio and cost ratio



Material in one lin foot of handrail and wheelguard

	Concrete C.Y.	Reinforcing Lbs
Handrail	0.04	5.00
Wheelguard	0.059	1.70

Note 0.34 c.y. additional concrete necessary for 4 end posts

FIGURE 20

QUANTITY CURVES FOR STANDARD REINFORCED CONCRETE SLAB BRIDGES

Superstructure only

Class "A" Loading

Note Above quantity curves do not include handrail or wheelguard

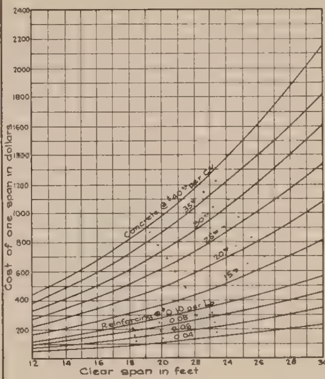


FIGURE 21-a

COST CURVES FOR ONE SPAN OF STANDARD SLAB BRIDGES

Superstructure only

Class "A" Loading 18' Roadway

Note: Above costs do not include handrail or wheel guard For quantities of material see fig 20

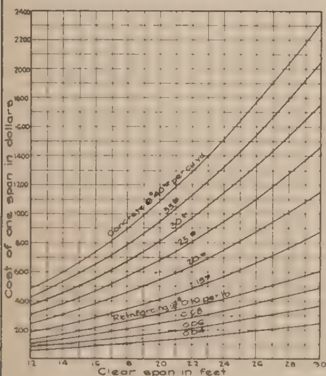


FIGURE 21-b

COST CURVES FOR ONE SPAN OF STANDARD SLAB BRIDGES

Superstructure only

Class "A" Loading 20' Roadway

Note Above costs do not include handrail or wheelguard For quantities of material see fig 20

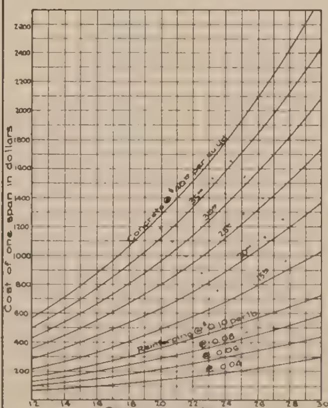


FIGURE 21 c

COST CURVES FOR ONE SPAN OF STANDARD SLAB BRIDGES

Superstructure only

Class "A" Loading 24 Roadway

Note Above costs do not include handrail or wheelguard For quantities of material see Fig 20

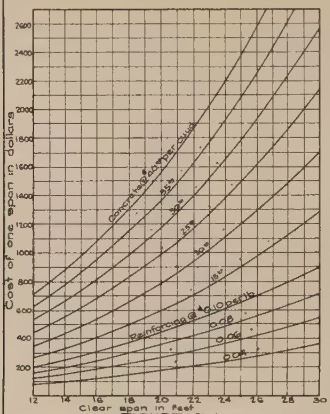


FIGURE 21 d
COST CURVES FOR
ONE SPAN OF
STANDARD SLAB BRIDGES
Superstructure only
Class "A" Loading 30' Roadway
Note Above costs do not include handrail or wheelguard
For quantities of material see Fig. 20

are not the same in this case owing to the fact that *form work* is much simpler for the slab type. As concrete materials become harder to secure and more costly to deliver at the site the saving in concrete on the *beam* types will offset to a greater extent the extra expense of form work and vica versa, so that the beam types show up to increasing advantage where concrete materials are expensive. The slab type is possibly a little easier to construct and may be built with less skilled supervision than the other types since the matter of shear keying between slab and beams, bonding of construction joints and general sequence of pour need not receive such close attention. This advantage, however, is hardly sufficient to materially effect unit costs.

Article 7.—Reinforced Concrete Through Girders

Figure 34 is a typical section for construction of this type while Figure 35 contains a series of quantity curves for roadway widths of 18, 20 and 24 feet. Cost curves based on the above quantities are given in Figures 36a, 36b and 36c. Figures 37 and 37a are comparisons in quantities and vertical clearance obstruction between this type and the multiple beam and girder types for roadway widths of 18 feet and 24 feet. From a study of these figures it is readily seen that the only advantage for this type lies in the matter of clearance. It is also observed that as the roadway widths are increased the cost of this type of con-

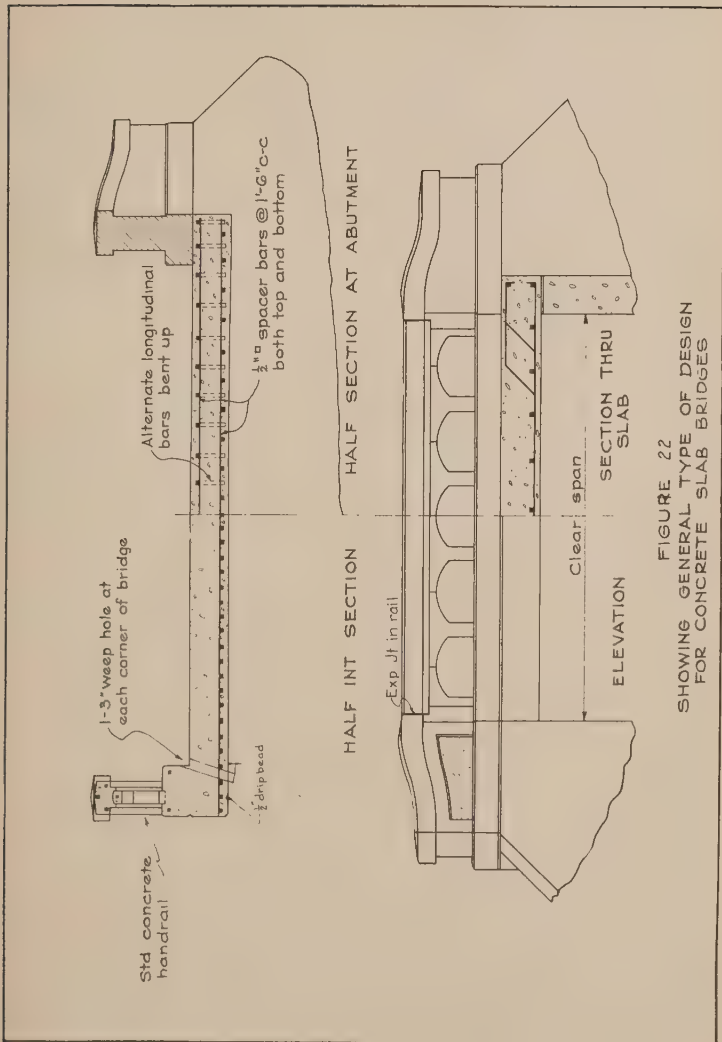


FIGURE 22
SHOWING GENERAL TYPE OF DESIGN FOR CONCRETE SLAB BRIDGES

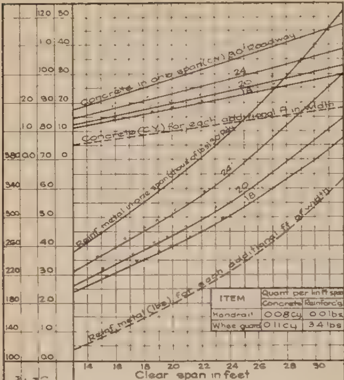


FIGURE 23
QUANTITY CURVES FOR
ONE SPAN OF
STANDARD R C MULTIPLE BEAM BRIDGES
Superstructure only Class 'A' Loading

Note Above curves do not include handrail or wheelguard

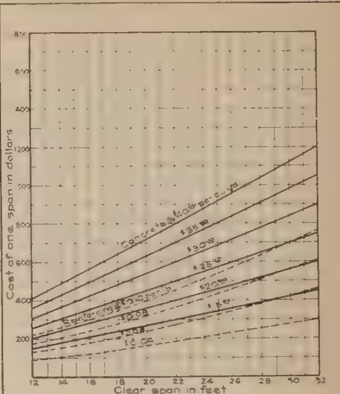


FIGURE 24-a
COST CURVES FOR
ONE SPAN OF
STANDARD R C MULTIPLE BEAM BRIDGES
Superstructure only
Class 'A' Loading 18' Roadway

Note Above costs do not include handrail or wheelguard
For quantities of material in handrail & wheelguard see fig

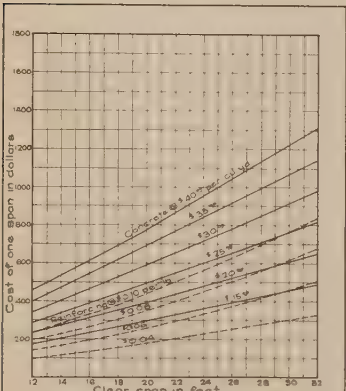


FIGURE 24-b
COST CURVES FOR
ONE SPAN OF
STANDARD R C MULTIPLE BEAM BRIDGES
Superstructure only
Class 'A' Loading 20' Roadway

Note Above costs do not include handrail or wheelguard
For quantities of materials in handrail & wheelguard see fig

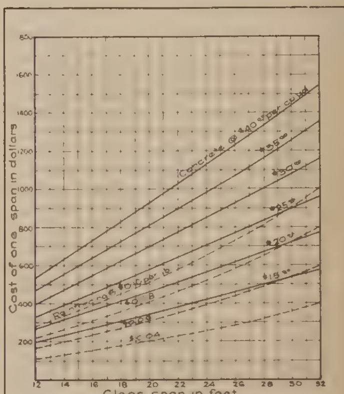


FIGURE 24-c
COST CURVES FOR
ONE SPAN OF
STANDARD R C MULTIPLE BEAM BRIDGES
Superstructure only
Class 'A' Loading 24' Roadway

Note Above costs do not include handrail or wheelguard
For quantities of material in handrail & wheelguard see fig

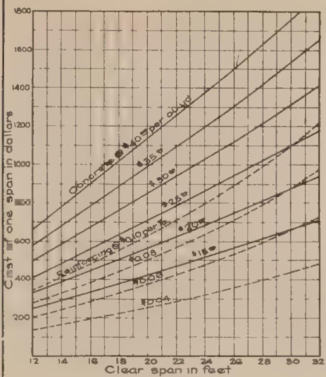


FIGURE 24-d
COST CURVES FOR
ONE SPAN OF
STANDARD R C MULTIPLE BEAM BRIDGES
Superstructure only
Class "A" Loading 30' Roadway

Note Above costs do not include handrail or wheelguard
For quantities of material for handrail & wheelguard see fig

struction increases much more rapidly than the other types of R. C. short span construction. Altogether this type has little to recommend it except clearance economy and even where the question of clearance is a paramount consideration, it may pay to raise the grade in order to secure requisite clearance for deck construction. Especially is this true of wide roadways.

SECTION II—LONGER SPAN SUPERSTRUCTURES

Article 8.—Uncovered Timber Trusses

Superstructure spans in this class divide themselves naturally into three groups as follows:

A-frame spans.
Pony or low truss spans.
High or through truss spans.

The timber A-frame design for class A loading is shown in detail in Figure 38 while Figure 39 is a photographic view of a structure built from this design over a dry waterspout gulch where flood debris carried by the stream during freshet stages necessitate a longer clear span than could be obtained by the adoption of trestle construction. Figure 40 is a standard design for a Class A low truss span. Figure 41 is a standard design for a high truss span of this same loading class. These designs are of an extremely high class *as regards details* being designed for long

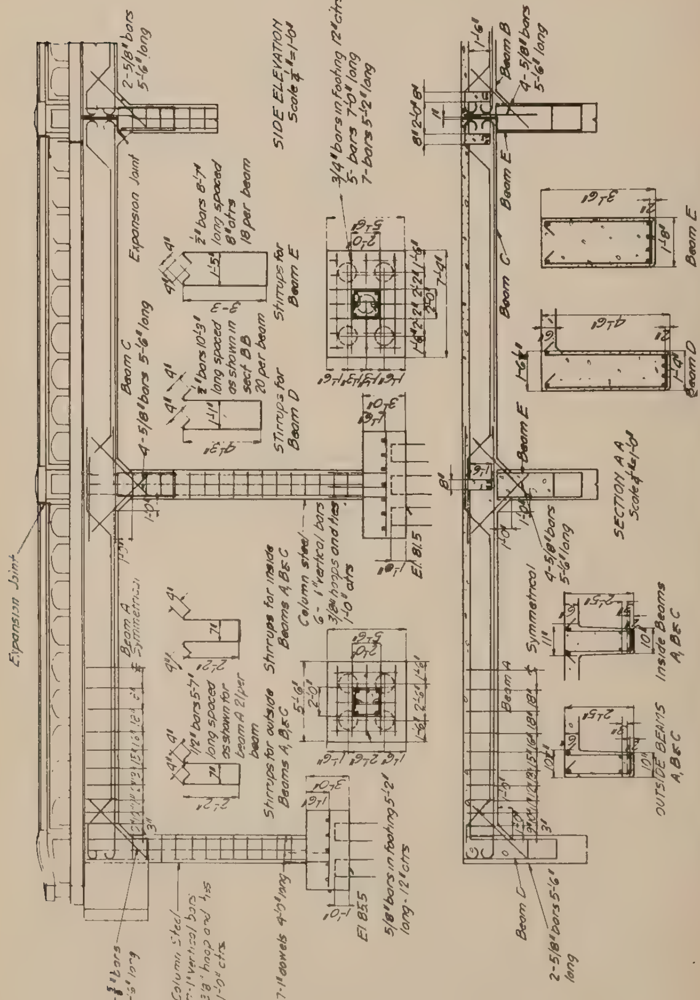


Figure 25. Elevation, typical R. C. multiple beam design, buried abutments.

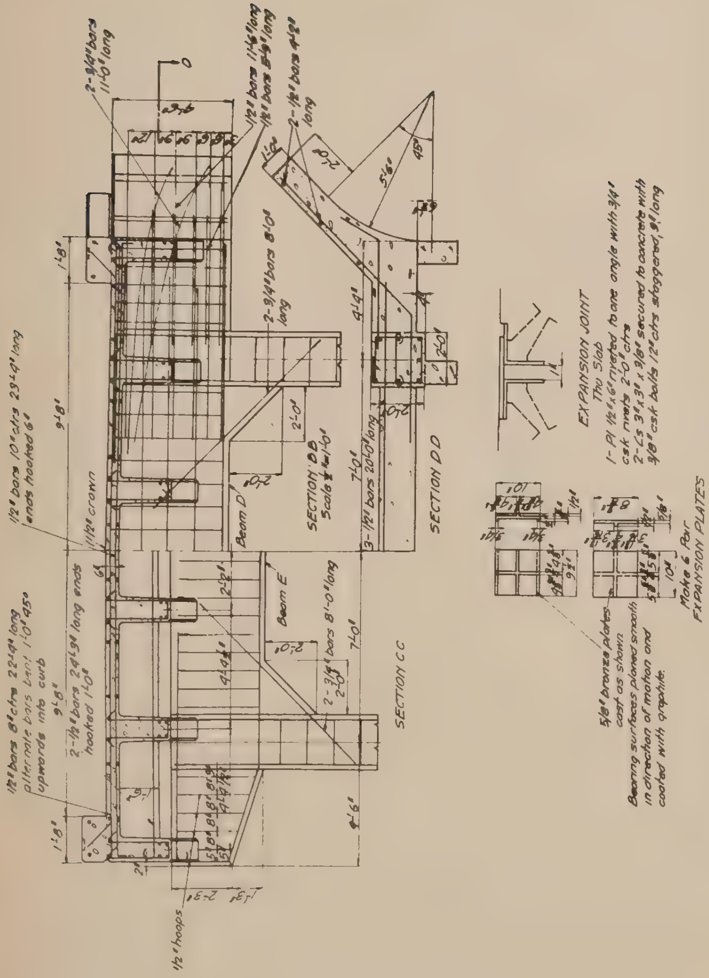


Figure 25a. Cross section, typical R. C. multiple beam design, buried abutments.

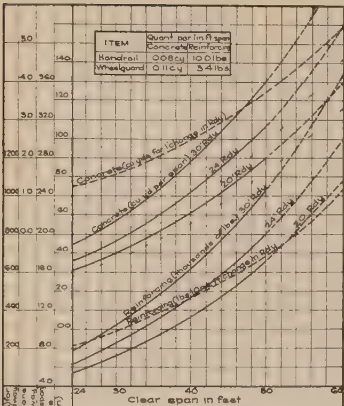


FIGURE 26
QUANTITY CURVES FOR STANDARD R.C. DECK GIRDER BRIDGES (3 GIRDERS)
Superstructure only

Note Above quantities do not include handrail or wheelguard. The above quantities include 2' haunch on each beam and a 15" abutment from floor slab to bottom of haunches.

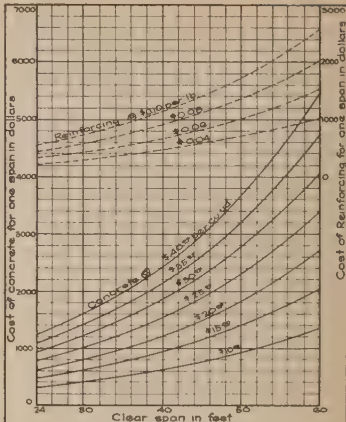


FIGURE 27-a
COST CURVES FOR STANDARD REINFORCED CONCRETE DECK GIRDER BRIDGES (3 GIRDERS)
Class 'A' Loading 20' Roadway
Superstructure only

Note The above quantities include two foot haunches on each beam and a 15' abutment from floor slab to bottom of haunch. They do not include handrail or wheelguard.

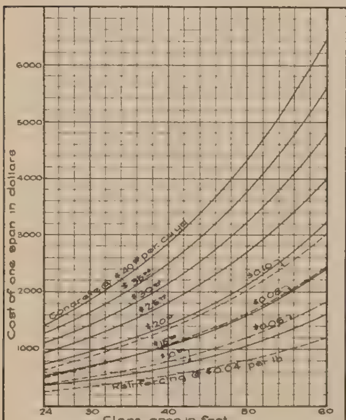


FIGURE 27-b
COST CURVES FOR STANDARD REINFORCED CONCRETE DECK GIRDER BRIDGES (3 GIRDERS)
Class 'A' Loading 24' Roadway
Superstructure only

Note The above quantities include two foot haunches on each beam and a 15' abutment from floor slab to bottom of haunches. They do not include handrail or wheelguard.

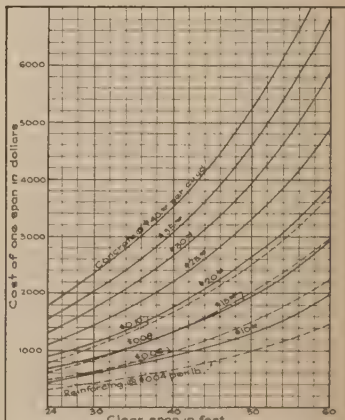


FIGURE 27-C
COST CURVES FOR STANDARD REINFORCED CONCRETE DECK GIRDER BRIDGES (3 GIRDERS)
Class 'A' Loading 30' Roadway
Superstructure only

Note The above quantities include two foot haunches on each beam and a 15' abutment from floor slab to bottom of haunches. They do not include handrail or wheelguard.

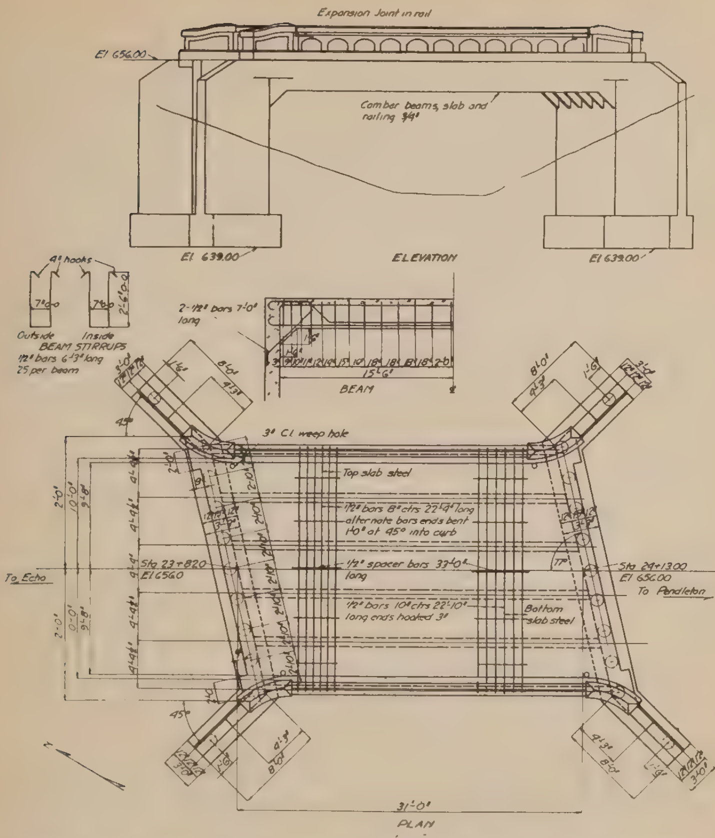


Figure 28. Typical R. C. multiple beam design, short wing abutments.

time service and with a view to lowering maintenance costs to the smallest possible figure. It will be noted that all joint connections employ cast iron angle blocks to eliminate decay at and around the joints to the maximum possible degree. All splices are of metal and positive in action. The details shown in these figures are self-explanatory. Figure 42 is of a standard design for a high truss span for Class B service. Not only is the truss design lighter but the joint details *omit the use of castings* and in several other ways the cost of details has been cut down. In localities where a supply of structural timber is not locally available, it is quite probable that it will not prove economical to construct timber

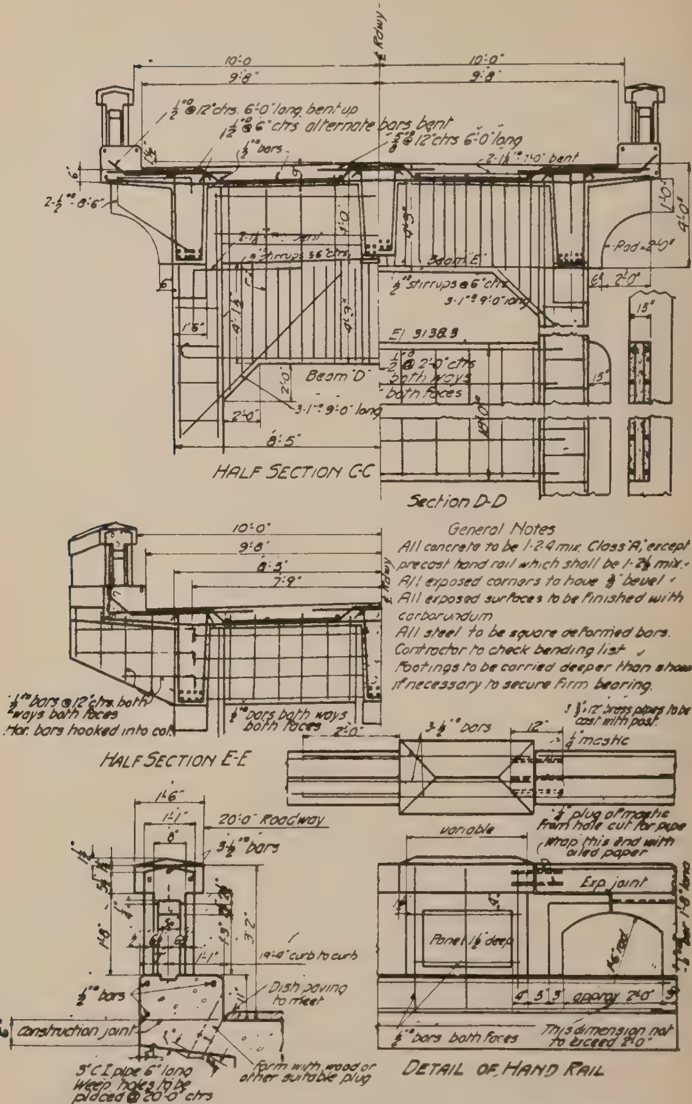


Figure 29. Typical cross section R. C. deck girder viaduct.

Figure 30. Typical reinforced concrete slab design. (Note low type handrail.)

Figure 31. Construction view of reinforced concrete deck girder design.

Figure 32. Typical deck girder design.

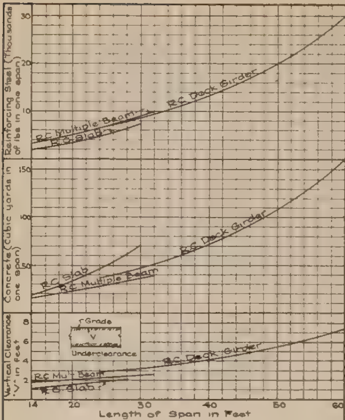


Figure 33
CURVES SHOWING
QUANTITIES AND VERTICAL CLEARANCES
FOR
VARIOUS TYPES OF CONCRETE BRIDGES
Class "A" Loading ~ 24' Roadway

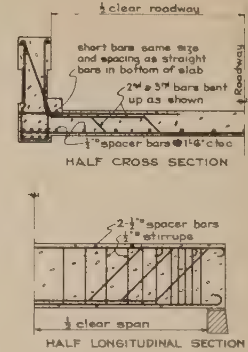


FIGURE 34
TYPICAL SECTIONS
OF
CONCRETE THRU GIRDER BRIDGES

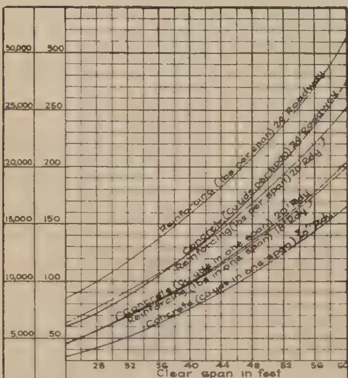


Figure 35
QUANTITY CURVES FOR
ONE SPAN OF
CONCRETE THRU GIRDER BRIDGE
Superstructure only ~ Class "A" Loading

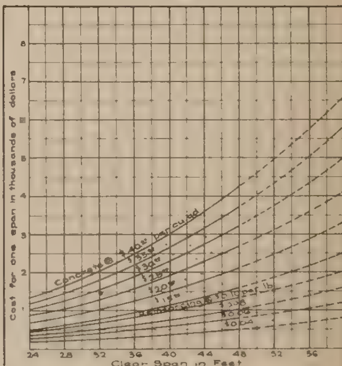


Figure 36a
COST CURVES FOR
ONE SPAN OF STANDARD
CONCRETE THRU GIRDER BRIDGE
Superstructure only ~ Class "A" Loading
18' Roadway

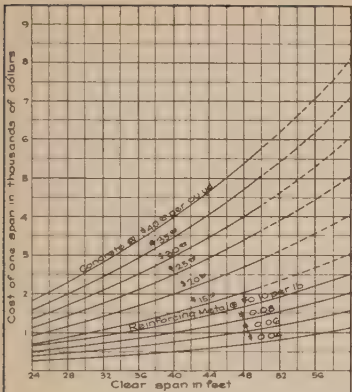


Figure 36 b

COST CURVES FOR
ONE SPAN OF STANDARD
CONCRETE THRU GIRDER BRIDGE
Superstructure only Class "A" Loading
20' Roadway

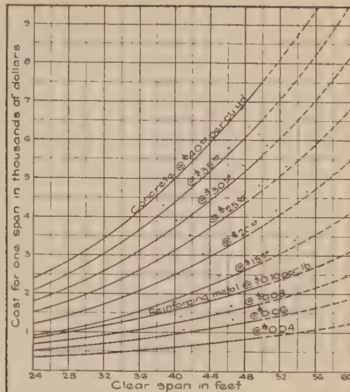


Figure 36 c

COST CURVES FOR
ONE SPAN OF STANDARD
CONCRETE THRU GIRDER BRIDGE
Superstructure only class "A" Loading
24' Roadway

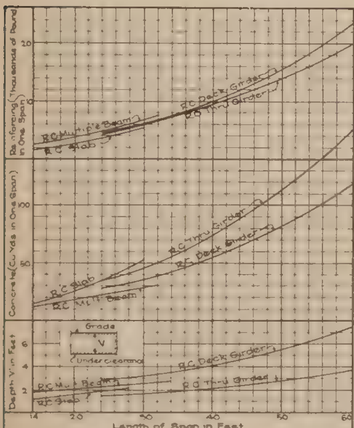


Figure 37-a

CURVES SHOWING
QUANTITIES AND VERTICAL CLEARANCES
FOR
VARIOUS TYPES OF CONCRETE BRIDGES
18' Roadway – Class "A" Loading

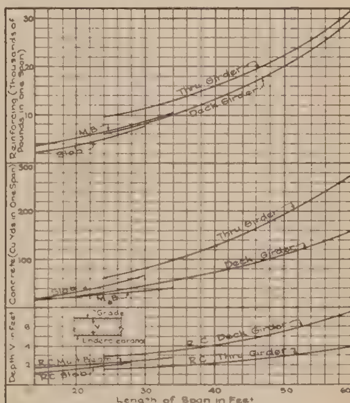


Figure 37-b

CURVES SHOWING
QUANTITIES AND VERTICAL CLEARANCES
FOR
VARIOUS TYPES OF CONCRETE BRIDGES
24' Roadway Class "A" Loading

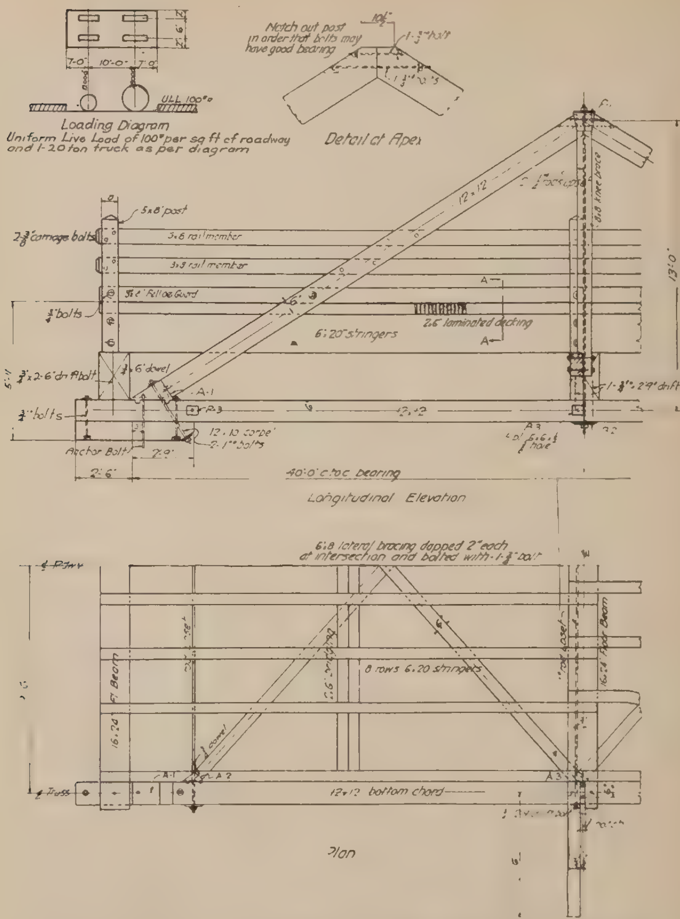


Figure 38. Plan and elevation, standard timber A-frame design.

spans to a higher standard than Class B since the heavier construction would show little economy in first cost over steel and would fall a long way behind the steel design in the matter of maintenance and renewal costs.

Figures 43 to 50, inclusive, are quantity and cost curves for the types above described, and are doubtless self-explanatory. Figures 51, and 52 are photographs of structures constructed to this type of standard

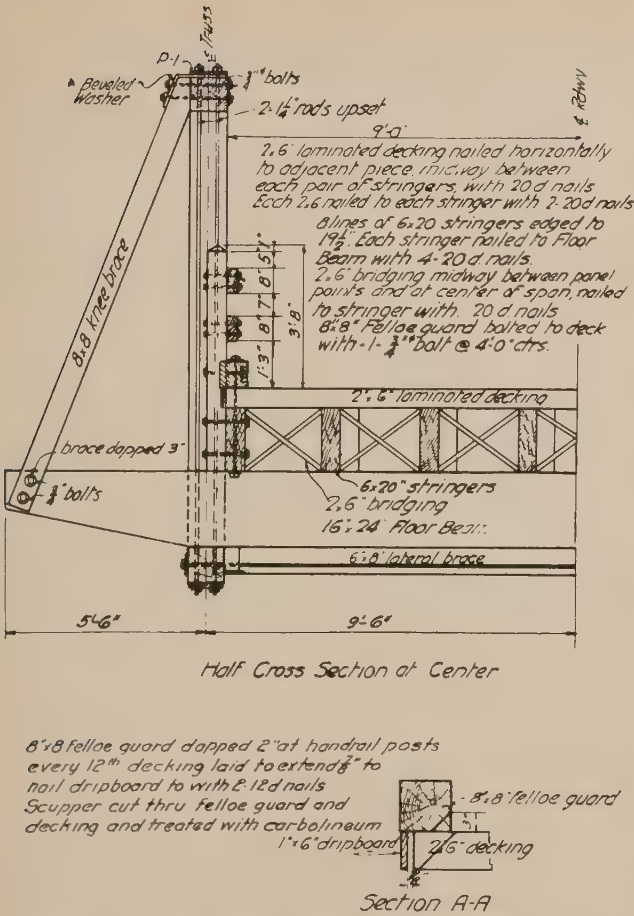


Figure 38a. Cross section, standard timber A-frame design.

The building of uncovered timber spans, except perhaps A-frames, is not warranted under ordinary conditions. In the humid regions, decay is rapid and in the arid sections trouble is experienced with warping and twisting out of bearing and with season checking so that, all in all, the service life is comparatively short and the maintenance cost high. (See Chapter V). Under ordinary conditions, therefore, such construction will not show to advantage in the economic analysis. For certain timbered sections of the west (and south), however, especially in localities very

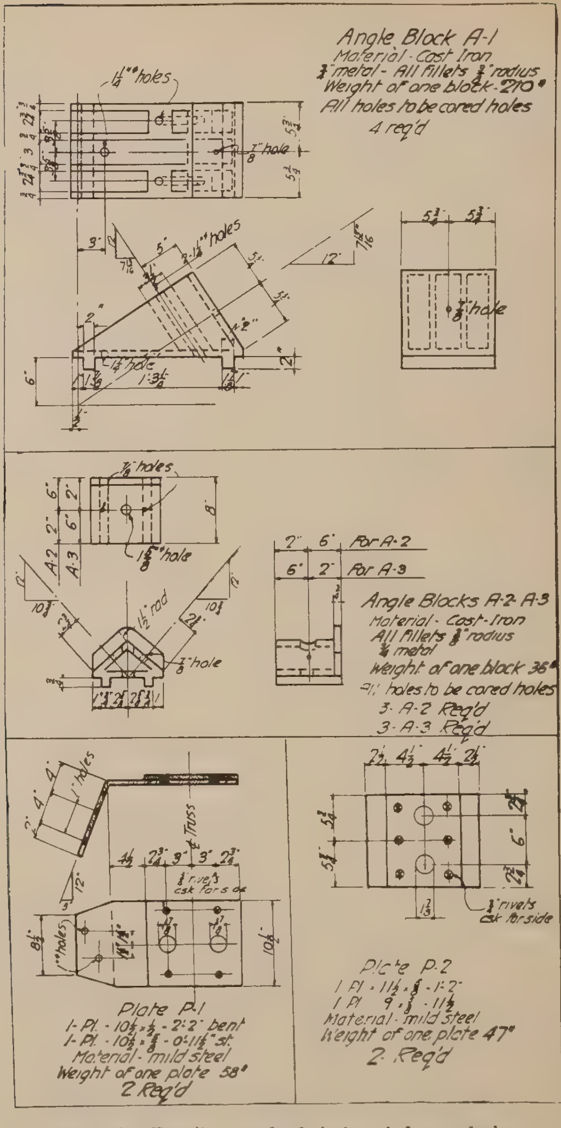


Figure 38b. Details, standard timber A-frame design.

remote from rail transportation, it is found that this type of construction may, in many cases, be employed to good advantage, opening up sections of country at a comparatively low bridging cost and utilizing such construction with the thought that replacements with steel or concrete may be made as the structures progressively deteriorate under service and as traffic density warrants. In this connection, however, it should be pointed out that, with the load concentrations and impact effects incident to present day highway transport, even timber construction must be of a high standard, not only as regards carrying capacity, but also as regards *rigidity and adequacy of details.* It needs but a comparison of the laminated 2″x6″ timber decking illustrated in Figure 38, for example, with the loose plank decking employed on the antiquated timber span shown in

Figure 39. Typical A-frame construction.

Figure 53, to bring out the difference in present and former transportation needs. Mention has already been made of the use of cast iron angle blocks and joint details (see Figures 38, 40 and 41). There is no question but that the use of these angle blocks adds to the life of the structure through the elimination of the ever present tendency for decay at or around the dapped or scarfed jointings. The framing with cast angle blocks is square, which in itself operates to insure a more even and tightly fitting bearing and less room for moisture to accumulate. From the quantity and cost curves for this type, however, it is observed that the use of castings adds a very considerable item of first cost expense. For example, let the 100-foot Howe truss span for which quantities are shown in Figure 47 be taken:

Lumber, 44 MFBM at $55.00 per M in place =	$2,420.00
Structural metal and hardware, 16,000 lbs. at .08 in place =	1,280.00
Castings, 12,000 lbs. at .08 in place =	960.00
	$4,660.00

Percentage cost of castings, $\frac{960}{4660} = 21\%$

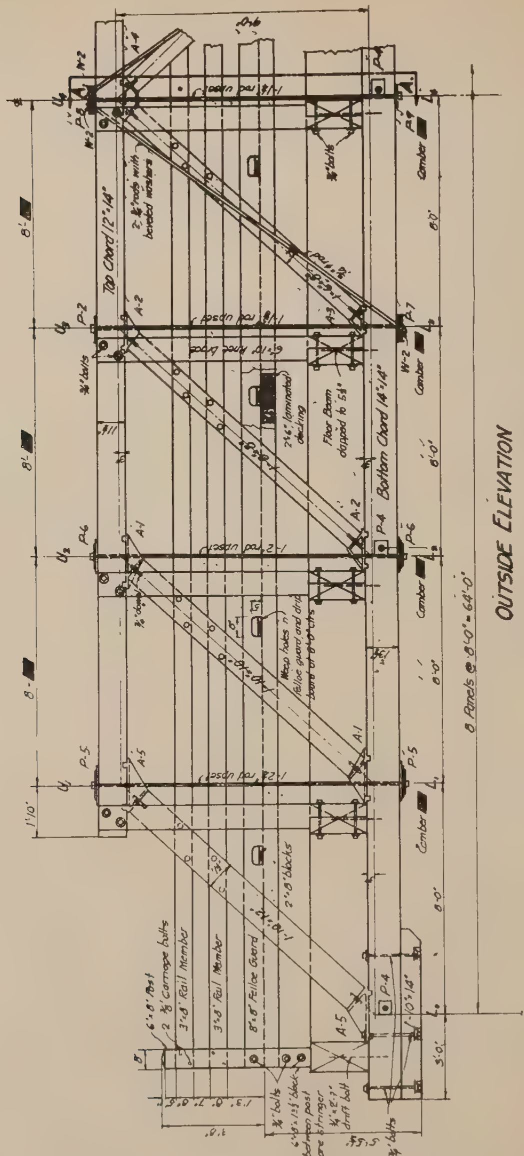


Figure 40. Standard low truss timber span.

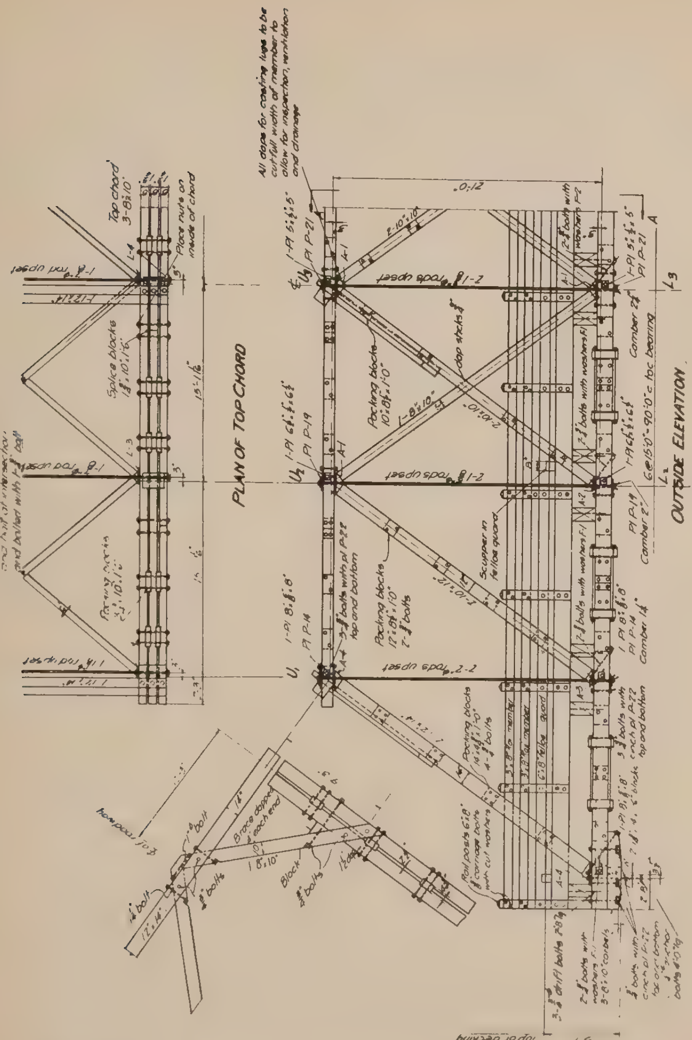


Figure 41. Typical high truss timber span.

The omission of the angle block castings would not reduce the cost of the span by 21% since a portion of this amount would need to be used for additional structural metal and for the additional expense of joint framing. It is probable, however, that the use of castings will

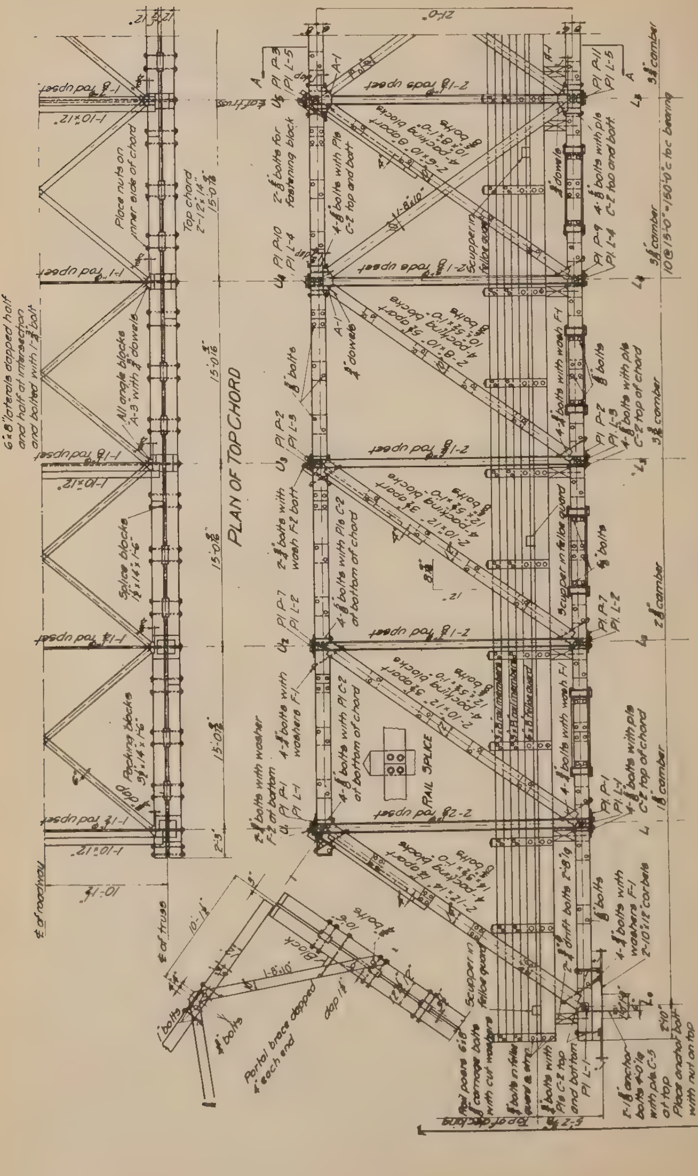


Figure 42. Typical 150-foot Howe truss, 18 foot roadway.

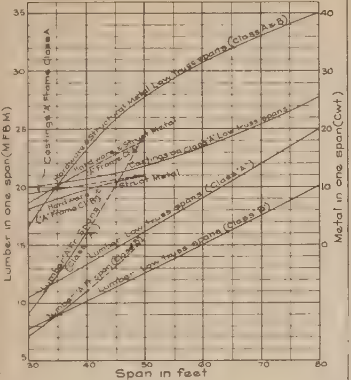


Figure 43

CURVES SHOWING QUANTITIES FOR LOW WOODEN TRUSS SPANS AND "A" FRAMES

Class 'A' & 'B' Loading 18' Roadway

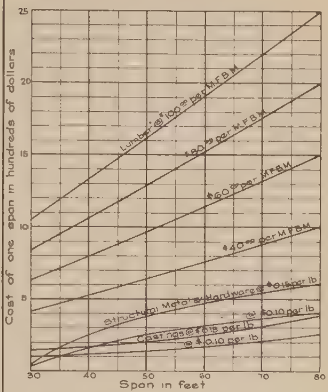


Figure 44

COST CURVES FOR PONY TRUSS SPAN BRIDGES

Class 'A' Loading — 18' Roadway

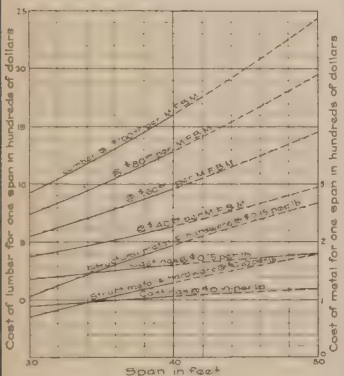


Figure 45

COST CURVES FOR "A" FRAME BRIDGES

Class 'A' Loading 18' Roadway

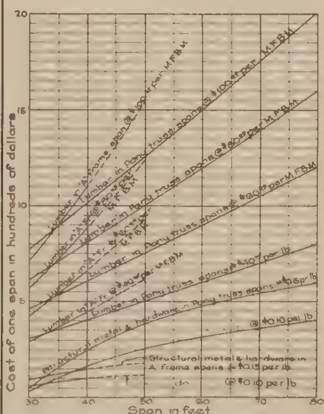


Figure 46

COST CURVES FOR LOW WOODEN TRUSS SPAN BRIDGES

Class "B" Loading 18' Roadway

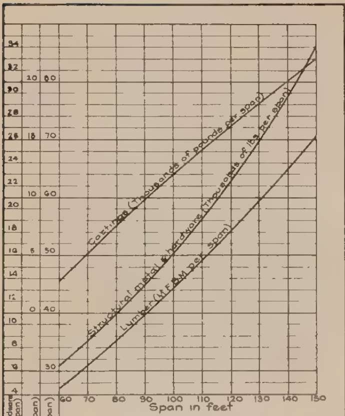


Figure 47
QUANTITY CURVES FOR
ONE SPAN OF
STANDARD HOWE TRUSS SPAN
Without housing
Class "A" Loading 18' Roadway

Note Structural metal & hardware includes plates, rods, splice bars, nails and bolts

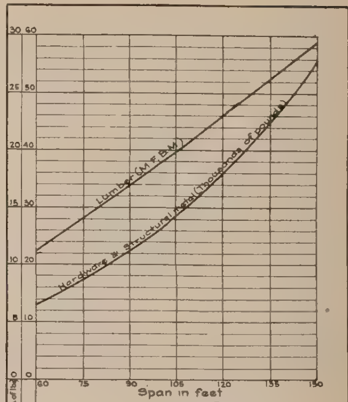


Figure 48
QUANTITY CURVES FOR
ONE SPAN OF
STANDARD HOWE TRUSS BRIDGES
Without housing
Class "B" Loading 18' Roadway

Note Structural metal and hardware includes plates, rods, splice bars, nails and bolts

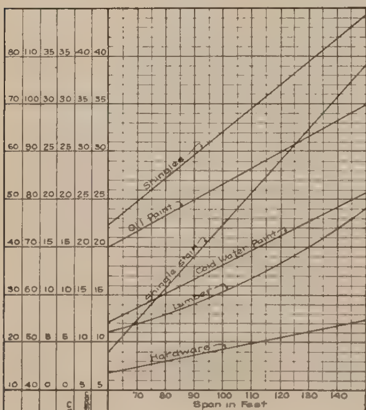


Figure 48-a
QUANTITY CURVES FOR
STANDARD HOUSING
FOR ONE SPAN OF HOWE TRUSS
Class "A" or "B" Loading 18' Roadway

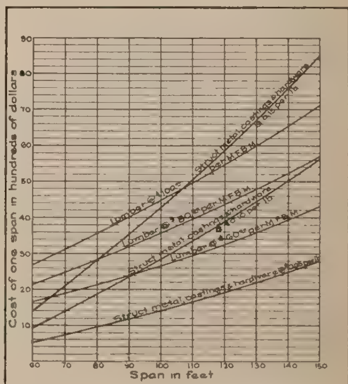


Figure 49
COST CURVES FOR
STANDARD HOWE TRUSS BRIDGES
Without Housing
Class "A" Loading 18' Roadway

Note - Structural metal and hardware includes plates rods, splice bars nails & bolts
Lumber includes paints & preservative

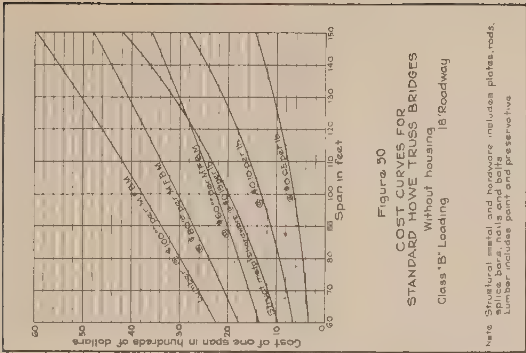


Figure 51. Typical design using timber pony truss spans.

Figure 52. Typical timber truss design, unhoused.

Figure 53. The timber bridge of a former generation—totally inadequate for modern traffic.

increase the superstructure cost 15%. For Class A designs (heavy loadings) it is probable that certain individual timbers in the span will be renewed from time to time as decay is observed until perhaps many of the truss members have been renewed. Since the joint castings operate to greatly reduce the cost of timber renewals and since they can be used over and over again with new timbering, their use is probably warranted. For Class B structures the program is generally that of carrying the timber structure at minimum expense during a shorter service period and later replacing it with steel or concrete construction. For this reason cast angle blocks appear a rather unwarranted expense. In view of the foregoing it will be noted that all timber truss construction illustrated herein which is to carry Class A loadings has been designed with cast angle blocks, while the Class B trusses are designed with the ordinary dapped or scarfed joints. The quantity and cost tables are prepared in accordance with the above practice.

This chapter is doubtless not the place for any elaborate discussion of truss details; however, it would seem apropos to briefly call attention to certain of the features embodied in the designs illustrated in Figures 38, 40, 41 and 42, since these details effect the quantity and cost data to an extent fully as great as the loading capacity itself. Among these details particular mention may be made of the following:

1. Laminated timber decking:—2″x6″ decking has been used for the Class A and 2″x4″ for the Class B designs. The method of horizontal

Figure 54. Modern type of housed timber span in comparison with the earlier type which it has replaced.

nailing is such as to tie the entire deck into one rigid and connected whole, the use of S1E pieces results in a very smooth and easy riding surface, while the "edge grain," or vertical grain, exposed to traffic results in a slow and even wear and a longer service life. Particular attention is called to the drip board and scupper arrangement for the protection of the ends of the decking pieces. Another detail which is even better than the laminated decking is the employment of 4″x6″ or 6″x6″ tongue and groove decking pieces driven to tight bearing as the deck is laid. This virtually insures a deck which will deflect evenly and act as a rigid and connected whole. The laminated decking is dependent for its integrity

Figure 55. Typical housed timber truss designed for modern traffic service. Note ventilation and lighting through to highway.

Figure 56. Construction view of housed timber span used in heavily timbered locality.

upon the use of horizontal nails which under heavy traffic do sometimes show a tendency to work loose, to shear and to fail in bending. All of these tendencies are corrected in the tongue and groove type which type, therefore, finds special adaptation where a bituminous pavement surface is to be placed over the deck, as the formation of cracks in the pavement surface is much reduced thereby.

2. Heavy bolted wheel guard employed as a traffic protection and also to afford the necessary restraint against warping at the ends of the decking pieces.

3. Stringer ventilation.

4. Metal chord joints.

5. Cinch bolts or packing bolts throughout the truss members to insure against season checks, etc., etc.

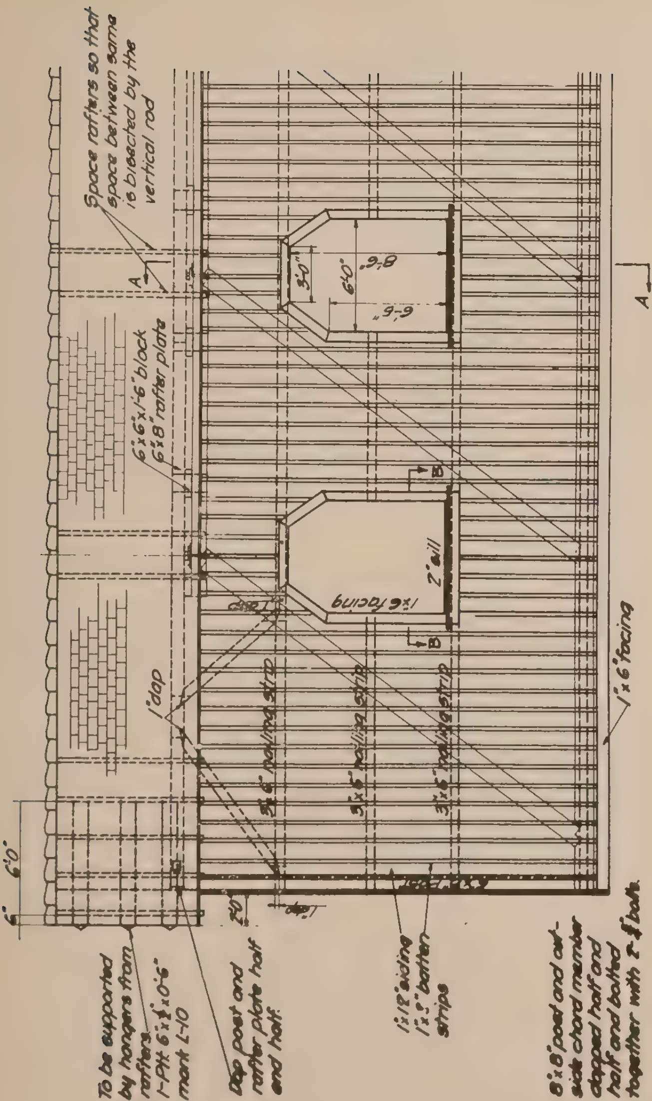


Figure 57. Elevation, Standard Howe truss housing details.

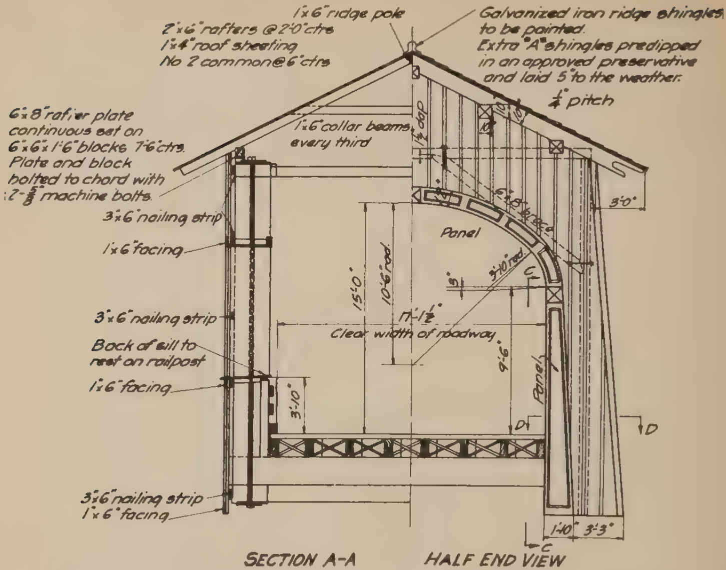


Figure 57a. Cross section, Standard Howe truss housing details.

6. Heavy hand railings for visibility and traffic protection.

7. Chamfered corbels to relieve secondary stresses at L_0 points. (See Figure 41.)

8. Other like details. (See Figures 38, 40, 41 and 42.)

The above list merely calls attention to certain of the details adopted in the designs shown with a view to the lowering of maintenance costs and the prolongation of the service life of the structure. All these details operate to increase first costs so that the costs given herein for such a type cannot be compared with the temporary timber truss types heretofore adopted in many sections of the country.

Article 9.—Housed Timber Truss Spans

It is a fact repeatedly demonstrated and generally accepted that the life of timber truss construction may be increased from 50% to 150% by housing in the truss members, thus protecting them from moisture and, hence, from decay.

The "covered bridge" of a former generation was an unsightly thing and the modern housed timber truss must not be confused with it or considered in the same class any more than the old time light pin connected "iron wagon bridge" on steel tubes is in the class of the modern steel

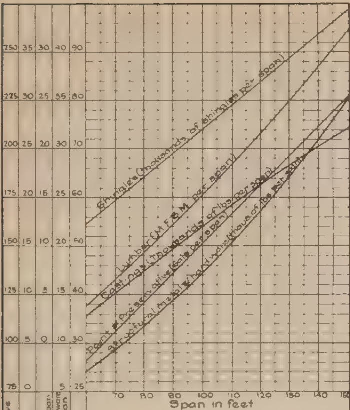


Figure 58

QUANTITY CURVES FOR ONE SPAN OF STANDARD HOWE TRUSS BRIDGES

With housing

Class "A" Loading 18' Roadway

Note Structural metal & hardware includes rods, plates, splice bars, nails and bolts

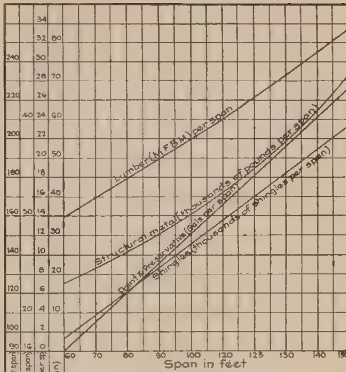


Figure 59

QUANTITY CURVES FOR ONE SPAN OF STANDARD HOWE TRUSS BRIDGES

With housing

Class "B" Loading 18' Roadway

Note Structural metal & hardware includes rods, plates, splice bars, nails and bolts

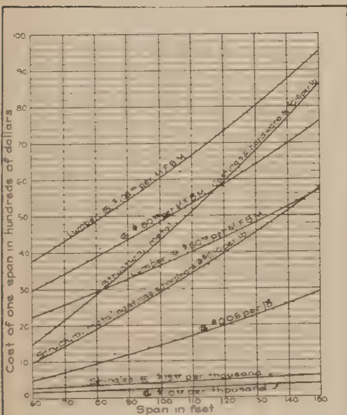


Figure 60

COST CURVES FOR STANDARD HOWE TRUSS BRIDGES

With housing

Class "A" Loading 18' Roadway

Note Structural metal and hardware includes plates, rods, splice bars, nails and bolts Shingles includes stain and preservative Lumber includes paint & preservative

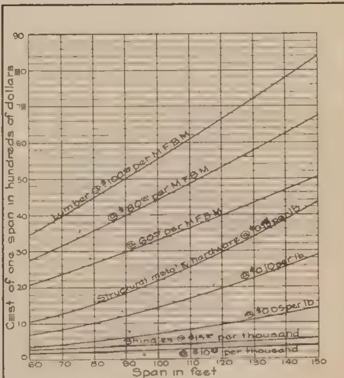


Figure 61

COST CURVES FOR STANDARD HOWE TRUSS BRIDGES

With housing

Class "B" Loading 18' Roadway

Note Structural metal and hardware include plates, rods splice bars, nails and bolts Shingles include stain and preservative Lumber includes paint and preservative

highway span. Figure 54, representing a portal view of the old and the new, aptly illustrates this difference. Figures 55 and 56 are photographic views of typical modern housed truss construction. Figure 57 is a design for suitable housing details (these details to be used in connection with the structural designs shown in Figures 41 or 42.) Cost and quantity curves for this type of structure are given in Figures 58, 59, 60 and 61.

Attention is directed to the attention paid to ventilation and lighting inside the truss and to the general details of construction. (See Figure 62.)

Article 10.—Riveted Steel Truss Spans

Superstructures in this class fall under three main groupings:

Low truss spans (50 to 100 feet).
Through truss spans (100 to 300 feet).
Special designs (spans up to 600 feet).

Figure 62. Interior of housed timber truss span, showing ventilation and lighting of housing.

Figure 63 is a standard design for a 60-ft. pony truss with what is designated as a 20-ft. roadway (in this case the clear distance between curbs has been taken at 19′ 8″ for certain other reasons). In this type of design there are no longitudinal joists, the concrete floor spanning directly between floor beams. Figure 64 is a 90-foot curved chord pony truss span having the same roadway width as above. Both of these designs are for Class A loading. Figure 64 shows the design of Figure 65

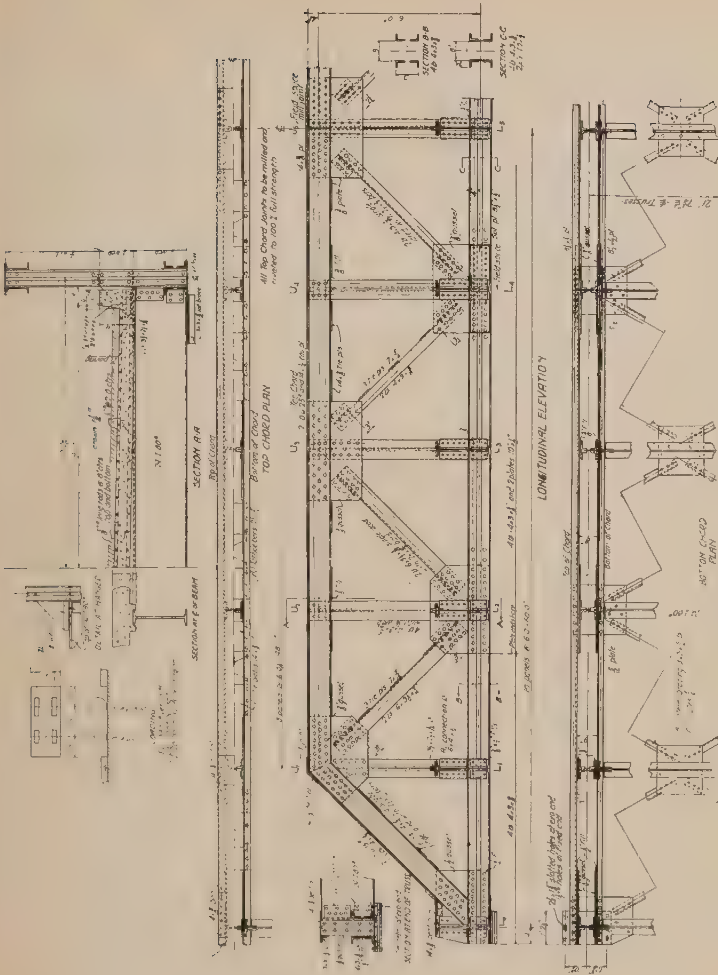


Figure 63. Typical 60-foot pony truss (straight chord).

as built. Figure 66 is a detail of a 90-foot curved chord pony truss span designed for a lighter loading. This design is arranged for either reinforced concrete or timber deck. In this connection it should be observed that in many instances there may be a demand for steel truss spans with decks cheaper in first cost than concrete in which case the laminated timber decking shown in Figure 66 is very good construction and will render

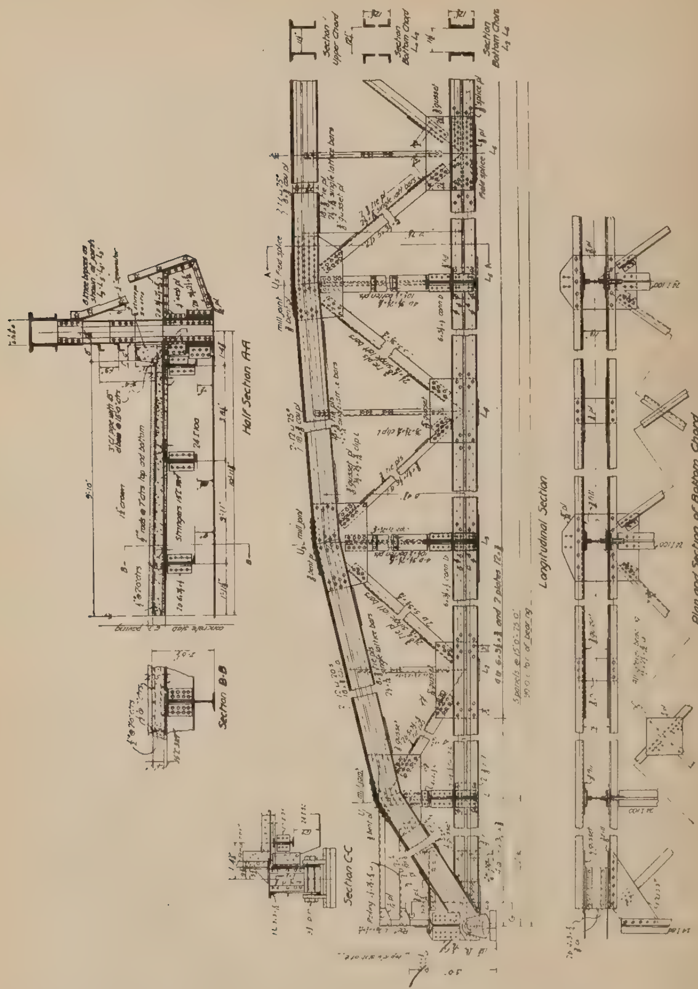


Figure 64. Typical 90-foot pony truss (curved chord).

excellent service for a period of from 8 to12 years, depending upon conditions as regards humidity, etc. In nearly every case, however, there is bound to be a future demand for a permanent floor for which reason the structural steel design should, in general, be proportioned to carry a future reinforced concrete deck.

Figure 65. Finished view low truss span shown in Figure 64.

Figures 67 and 68 are typical designs for riveted through truss spans of the same general class as above; both of these designs are for Class A loadings. Figure 67 illustrates the concrete floor type, with nominal 18 foot roadway (17′ 4″ clear between curbs; 18′ 2″ clear between handrails). Figure 68 employs a laminated timber deck design, 18 feet clear between wheel guards. Figure 69 is a photographic view of a completed structure of this type with reinforced concrete deck and approach spans.

Reference to the above figures will disclose the fact that the details are somewhat heavier than those typical of steel highway bridge construction in the past, this development being the natural outgrowth of the increase in speed and load concentration imposed upon our present day highway systems. Figure 64 illu trates the use of outrigger brackets which are fast being discarded in favor of the rigid floor beam connection shown in Figure 66, wherein the floor beam flange is coped and the web direct connected to the vertical truss members. It will be noted that heavy gusset plates have been used throughout and a minimum thickness of $\frac{5}{16}$ inches adopted for all the steel sections. Attention is also directed to the very rigid system of sway bracing used for the high truss designs with concrete flooring (Figure 67); to the dust covers used for the roller nests, and in general to the heavy detailing adopted throughout.

Figures 63 to 69, inclusive, represent a variety of design types not all of equal excellence. For example, the overhead bracing in Figure 67 is much superior to that for Figure 68; again the outrigger bracket of Figure 64 is very much inferior to the corresponding detail shown in

Figure 66. In view of the foregoing, the cost curves given in Figures 70, 71, and 72 have not been computed to cover any one type of design, but rather to indicate average weights between the range of choice ordinarily afforded in detailing.

Figure 70 is for low or pony truss spans for both Class A and Class B loadings. For spans between 70 and 100 feet, the weights for the Class B designs have been computed with the steel joists omitted for use where a temporary timber stringer system is to be considered. The table at the upper left hand corner of the figure furnishes data regarding the quantities of material necessary for different floor types used with the truss designs covered by the quantity curves below, the first column under the heading "lumber" giving the material needed for a timber joist or stringer system where the same is contemplated.

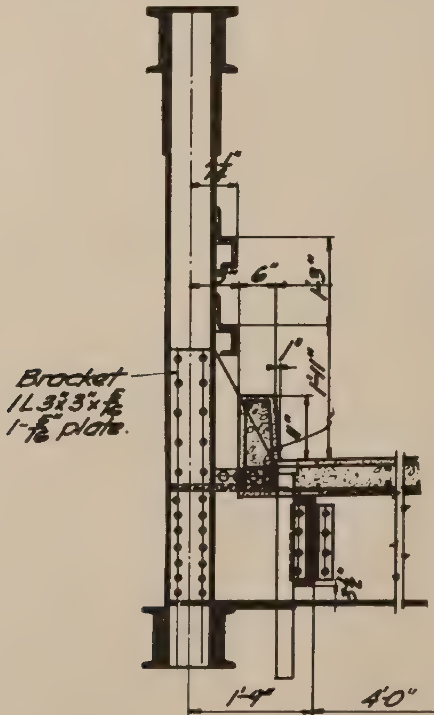


Figure 66. Bracing detail, standard steel pony truss.

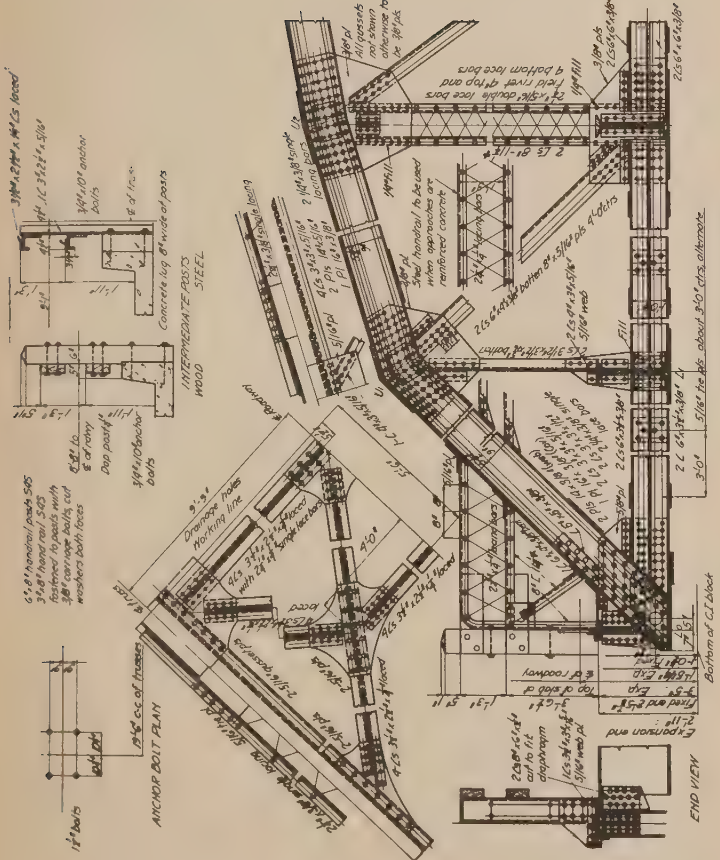


Figure 67. Longitudinal section, typical 200-foot riveted truss.

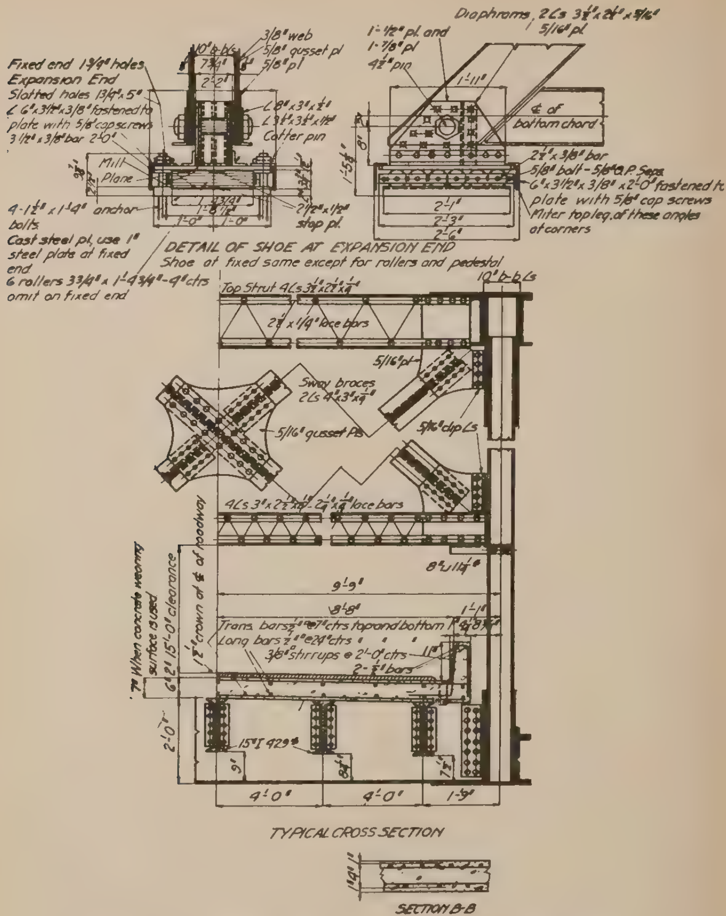


Figure 67a. Cross section, typical 200-foot riveted truss.

Figures 71 and 72 are for through truss spans of riveted joint design and for Class A and Class B loadings. All of the truss designs are computed for a future concrete floor and wearing surface as noted on the curves.

In passing it should be observed that the quantities given in the above figures are for designs embodying the most rigid and adequate system of detailing, all sections being completely developed and very careful atten-

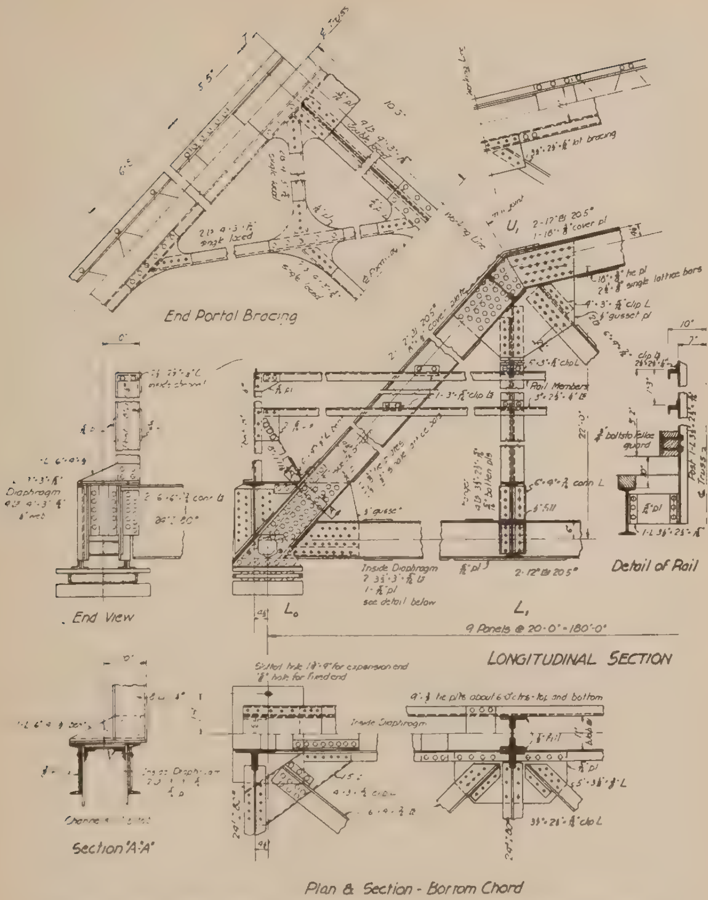


Figure 68. Elevation, standard 180-foot through steel truss.

tion paid to the securing of the utmost in rigidity at every point. It is possible to design structures with lighter details throughout under the same loading specification at a weight of about 85% of the weights given in the above curves. In view of the growth of highway traffic weight concentrations during the past decade and of the increase in average traffic speed with its attendant increase in impact, it is questionable if such a procedure is true economy.

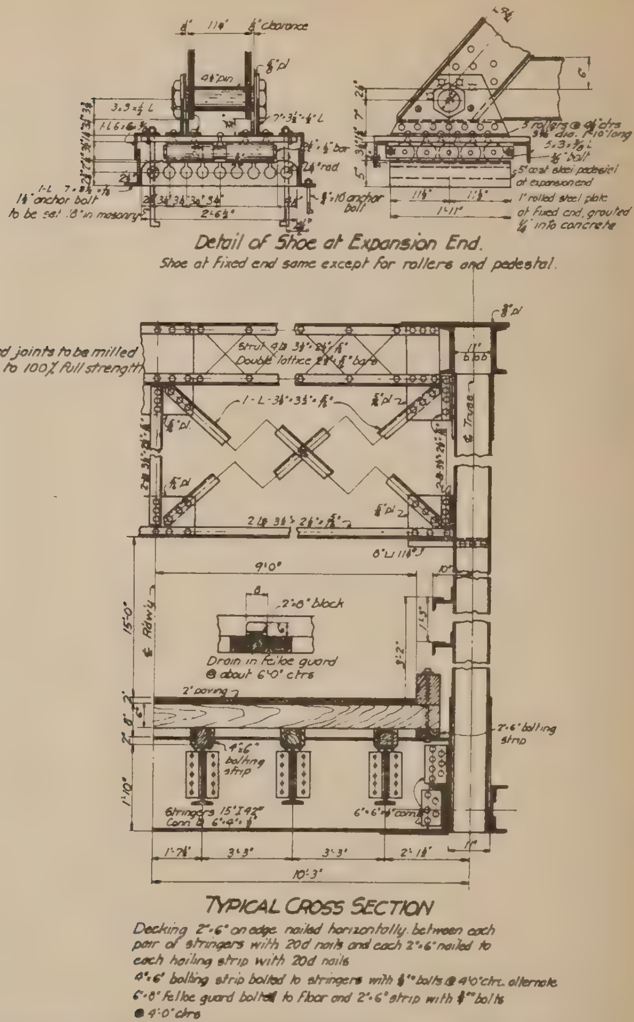


Figure 68a. Cross section, standard 180-foot through steel truss.

Article 11.—Pin Connected Steel Truss Spans

The quantity curves given in the foregoing article cover the riveted type of steel highway truss construction only. This type of design is

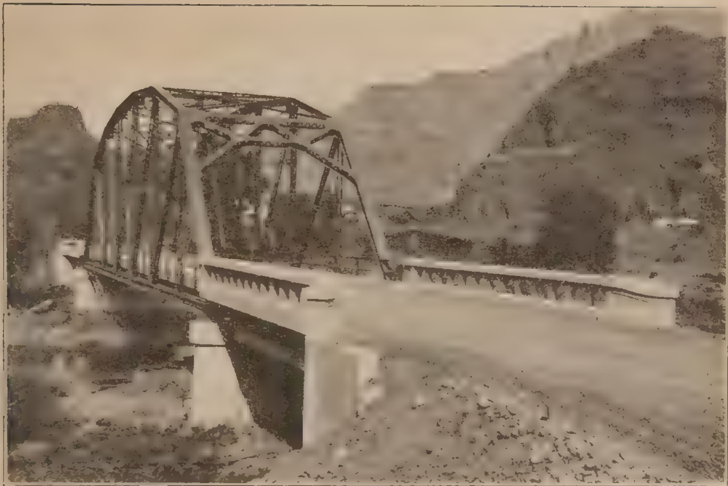


Figure 69. Typical through-riveted Pratt truss highway span with reinforced concrete deck and approach spans.

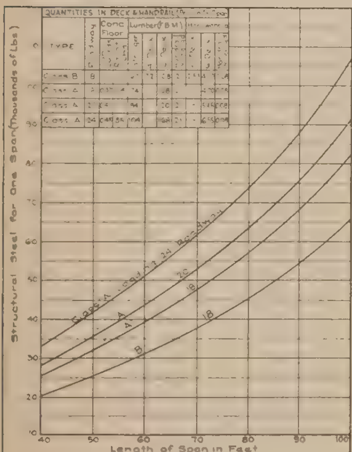


Figure 70
QUANTITY CURVES FOR STEEL
LOW TRUSS BRIDGES

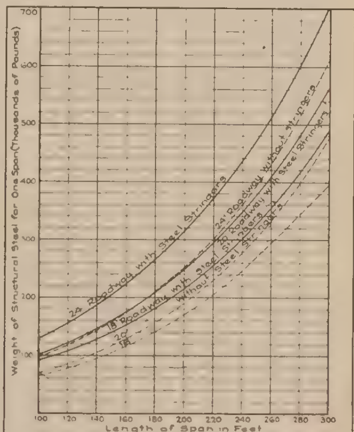


Figure 71
QUANTITY CURVES FOR STEEL
THRU TRUSS BRIDGES
Class "A" Loading

Note. All of the above trusses are figured heavy enough to carry a concrete floor

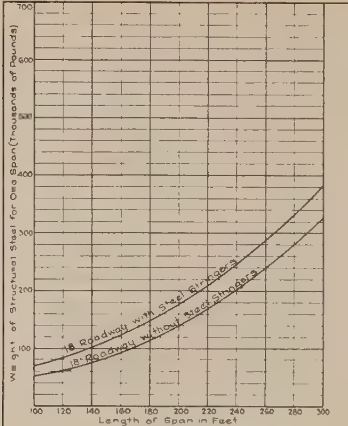
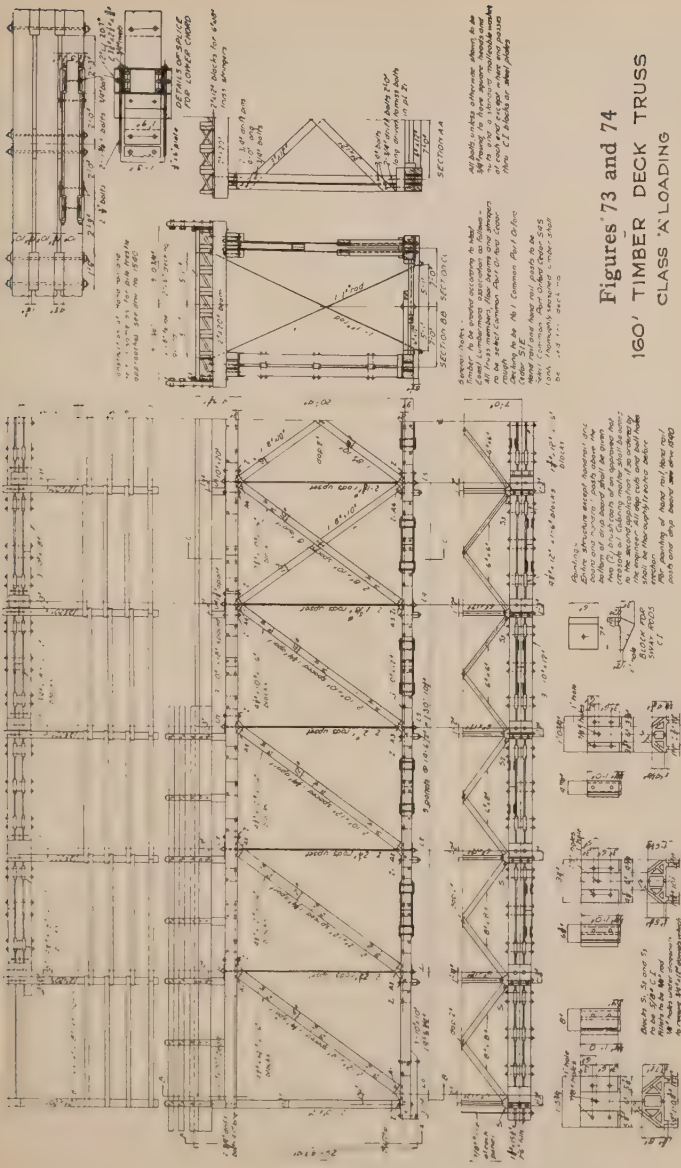


Figure 72
QUANTITY CURVES FOR
STEEL
THRU TRUSS BRIDGES
Class "B" Loading

Note All of the above trusses figured heavy enough to carry a 6" concrete floor

more rigid than the pin connected type although the latter is the more desirable from the standpoint of secondary stress elimination. For spans of less than 150 feet, nothing but riveted construction can be considered as truly adequate for modern highway transportation needs, principally on account of the lack of rigidity in the pin connected type. For spans above 150 feet the dead load becomes such a large percentage of the total load that the above objection has less weight than before, the objeection to pin connected construction decreasing as the span lengths are increased. For span lengths between 150 feet and 300 feet, pin connected construction may be used but it is a serious question whether the resulting construction will be as desirable. Above 300 feet, pin connected construction will probably answer the purpose very nicely.

For ordinary erection on false work, the pin connected type has a slight advantage both as regards cost and time of erection. In locations where flood, ice, or drift effects render it possible to carry false work only during a certain restricted season, it may be advantageous to consider the pin connected design type for this season. On the other hand, the riveted type lends itself much more readily to erection by the *cantilever method*, since the lower chords are generally capable of withstanding the compression stresses introduced thereby. All in all, the pin connected type has little to recommend it (except in special instances, as above noted) for span lengths below the 300-foot limit.



Figures 73 and 74
160' TIMBER DECK TRUSS
CLASS 'A' LOADING

The data for steel truss spans given hereinabove are for simple Pratt truss spans without subpanelling. No data have been given for "Petit" trusses or for other subpanel designs, as these are not generally used for span lengths below 250 feet and in the majority of cases are of special design. For spans above 400 feet in length, the distance c. c. trusses will generally be greater than that required for an 18-foot roadway, on account of the minimum ratios of width to height required by the specifications.

In view of the foregoing, the data for subpanelled and long span designs will be reserved for discussion later on in this chapter.

Article 12.—Timber Deck Trusses

A truss span 160 feet in length designed for Class A loading is shown in detail in Figures 73 and 74. This design employs the cast iron angle block type of joint detail and metal splices are employed throughout. Figure 75 is a construction view of a Class B timber deck truss span.

Housing is sometimes employed for this type of construction, however, its use is so rare that the quantity curves given in Figures 76, 77 and 78 are for the unhoused trusses only. The advantages of deck truss construction over like construction of the through type are (1) increased rigidity because of the fact that it is possible in this type to employ an adequate system of sway bracing at each panel point, (2) a saving in material in floor beams and (3) (in the case of timber construction) a greater protection of the timber against the weather. The principal disadvantage of this type lies in the increased requirement for clearance between grade and highest ice, drift, or flood elevation, as the case may be.

Figure 76 contains quantity curves for timber, structural metal and hardware and for castings for timber deck spans, Class A loading and 20-foot roadway in span lengths from 40 feet to 150 feet, which latter figure represents about the upper limit in span length for timber truss construction which may be built with any economy. Longer timber spans are sometimes used, but the difficulty in splicing the tension chords and in holding the truss to camber renders their use of doubtful propriety. Figure 77 is for Class A loading and an 18-foot roadway and Figure 78 is for Class B loading designs.

Article 13.—Steel Deck Truss Spans

A typical design for this type of structure, with timber deck and railing, is shown in Figure 79. The design illustrated in this figure contemplates the use of a 6″ laminated timber deck. The details of the stringer system, deck and railing are similar to those shown in Figures 2 and 3 for the standard trestle superstructure. Figure 80 is typical of

this class of construction when used with concrete deck and rail. The general type of structural detail used throughout these designs is much the same as has been adopted for the through truss designs shown in Figures 63 to 69. All metal has been designed with a five-sixteenths inch minimum thickness and special attention has been paid to rigid bracing, anchorage for floor joists, etc., etc. The design shown in Figure 80 omits the use of a stringer system, the concrete floor slab spanning directly the distance between floor beams. Figure 81 is a photographic view of a structure completed in accordance with this type of design.

The deck design types above described are for shallow truss construction. For the longer spans deep trusses are employed, as indicated in Figure 82.

The deck truss is a more rigid type of construction than the through

Figure 75. Construction view of timber deck truss design.

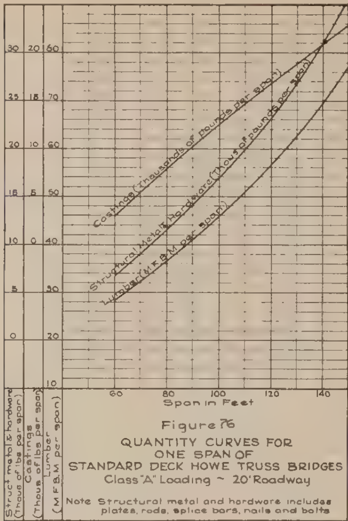


Figure 76
QUANTITY CURVES FOR
ONE SPAN OF
STANDARD DECK HOWE TRUSS BRIDGES
Class "A" Loading ~ 20' Roadway

Note Structural metal and hardware includes plates, rods, splice bars, nails and bolts

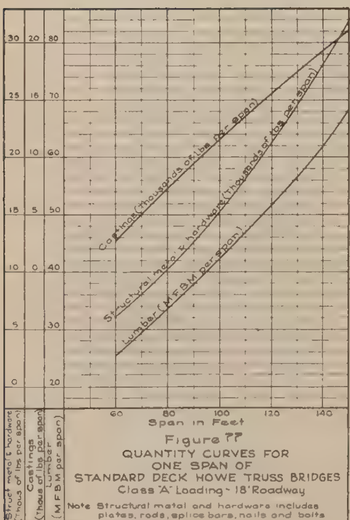


Figure 77
QUANTITY CURVES FOR
ONE SPAN OF
STANDARD DECK HOWE TRUSS BRIDGES
Class "A" Loading ~ 18' Roadway

Note Structural metal and hardware includes plates, rods, splice bars, nails and bolts

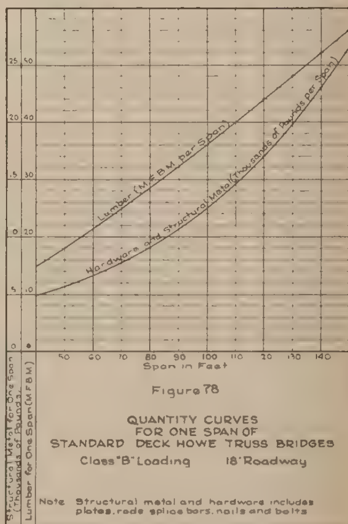


Figure 78

QUANTITY CURVES
FOR ONE SPAN OF
STANDARD DECK HOWE TRUSS BRIDGES
Class "B" Loading 18' Roadway

Note Structural metal and hardware includes plates, rods splice bars, nails and bolts

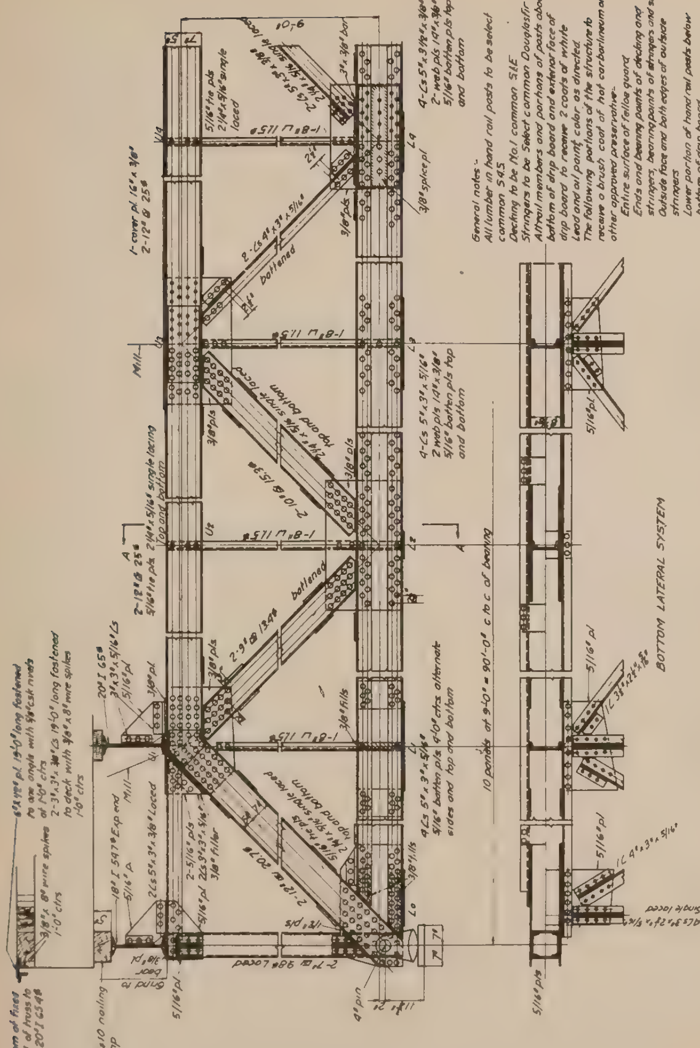


Figure 79. Elevation, typical 90-foot steel deck span.

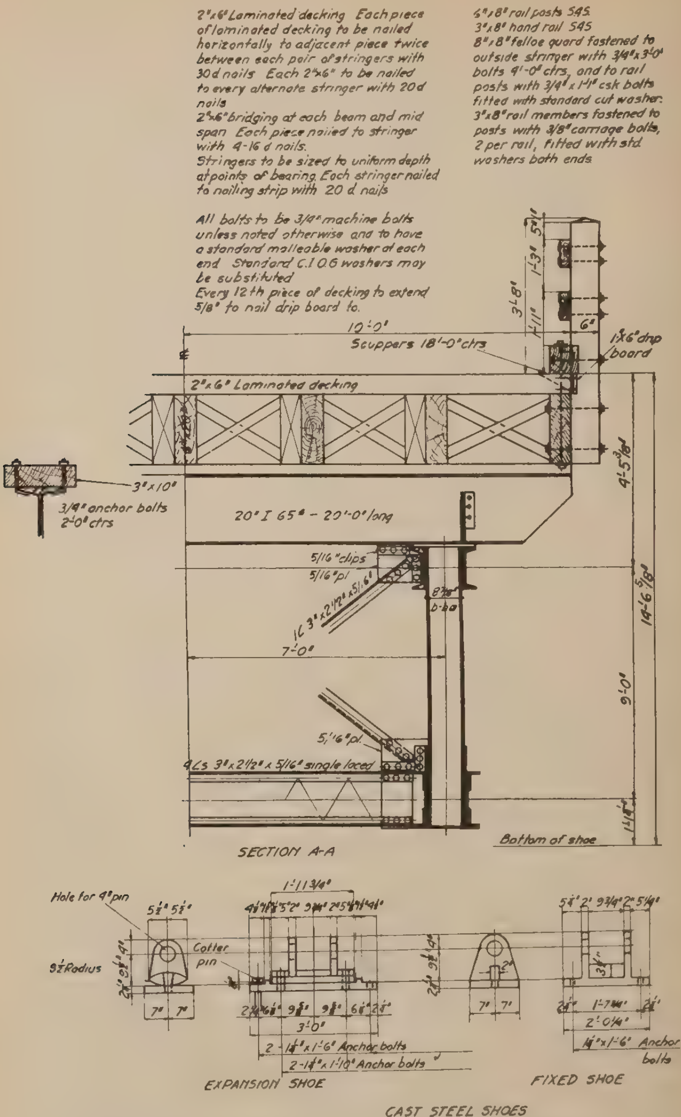


Figure 79a. Cross section, typical 90-foot steel deck span.

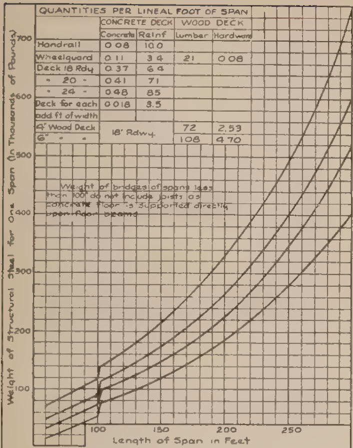


Figure 83
QUANTITY CURVES
SHOWING
WEIGHTS OF STEEL
DECK TRUSS BRIDGES

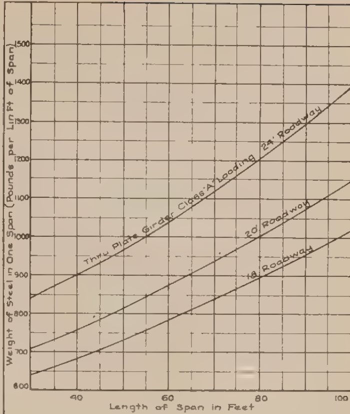


Figure 85
QUANTITY CURVES
FOR
THRU PLATE GIRDER BRIDGES
Class "A" Loading ~ Concrete Deck

Figure 84. Typical deck plate girder design. Reinforced concrete deck and rail.

The structural metal in deck truss construction is better protected than in the through type of design, but generally less accessible for inspection. The appearance of the deck design, especially with concrete railing, is much superior to that of the through design, the clear sight distance for traffic approaching the bridge is generally better and for wide roadways the use of the deck type of design is to be preferred for the reasons enumerated in Chapter II.

It is impossible to plot any quantity curves which will accurately represent the weights of deck truss spans in general, inasmuch as different design arrangements vary greatly in detail. Figure 83 represents average weights for the design types hereinabove described for roadway

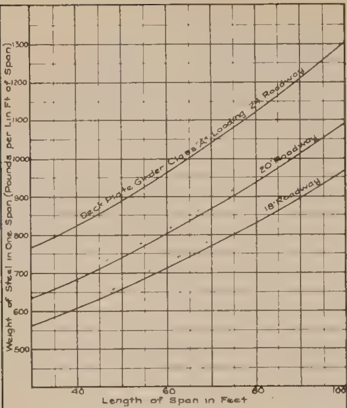


Figure 86
QUANTITY CURVES
FOR
DECK PLATE GIRDER BRIDGES
Class "A" Loading ~ Concrete Deck & Rail

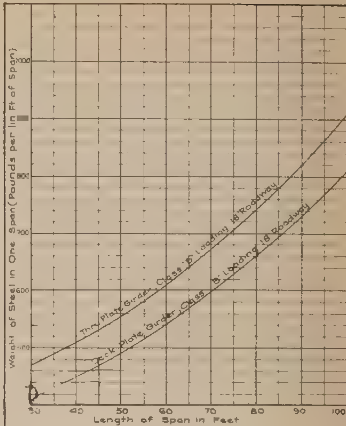


Figure 87
QUANTITY CURVES
FOR
THRU & DECK PLATE GIRDER BRIDGES
Class "B" Loading ~ 18' Roadway
Wood Deck

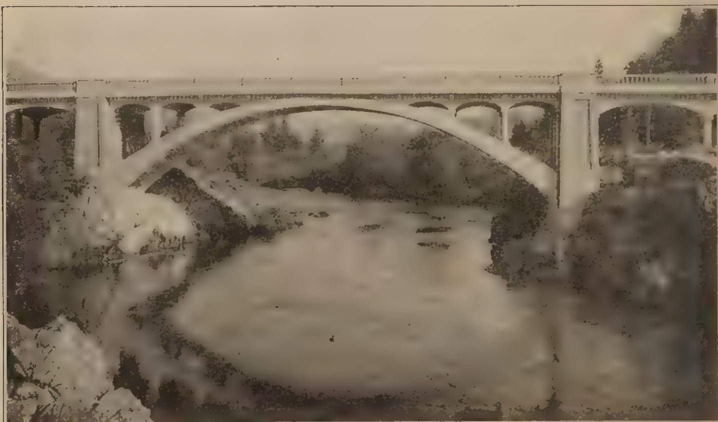


Figure 88. Arch span (open spandrel type) with abutment costs reduced, owing to the presence of steep, rocky river banks.

widths of 18, 20 and 24 feet and for span lengths between 50 feet and 300 feet.

Article 14.—Steel Deck Plate Girder Spans

Figure 84 is a typical design of this character, with concrete deck and

railing. This type is in direct price competition with the deck truss type. In general the weight of structural metal will be slightly greater than for a deck truss of like dimension. On the other hand, the pound price for metal will be slightly less for girder than for truss construction. This type of design is also in price competition with the reinforced concrete deck girder type described in Article 6 for spans up to 60 feet in length. In Figure 84 it will be noted that reinforced concrete deck girder construction has been carried out to a point where the necessary span length (determined by considerations of waterway adequacy) became so great as to necessitate the use of steel plate girder construction.

One of the chief advantages for this type is the possibility of a pleasing architectural treatment. In the design shown in Figure 84 the steel girders have been painted to match the concrete, so that the entire structure presents a harmonius color scheme.

Article 15.—Steel Through Plate Girder Spans

This type of steel plate girder design is suitable only where clearance requirements will not permit the use of the deck type. The appearance of this type is greatly inferior to that of the deck plate girder, especially when the latter is designed to carry a reinforced concrete deck and rail. This type is in direct price competition with the low steel through truss type in exactly the same manner as in the case of the corresponding deck types. It is also in competition with the reinforced concrete through girder type for roadways of 18 or 20 feet, it being understood, of course, that an economic comparison takes into consideration maintenance and renewal costs.

Figures 85, 86 and 87 are curves representing average weights for plate girder construction, both through and deck, of the types hereinabove described.

Article 16.—Reinforced Concrete Arch Spans

This type of structure is practically the only type of reinforced concrete construction suitable for *long spans* and its field of utility is generally restricted to spans above 50 feet. Where shorter spans are permissible, the concrete arch is in first cost competition with the multiple span, beam or girder type or with concrete viaduct construction, as shown in Figure 120.

Where longer spans are a necessity, this type of construction is in competition, as regards total costs (including maintenance and renewals) with the various types of steel construction hereinabove described.

There are two types of arch design ordinarily employed for highway bridges, one designated as the *open spandrel* type and the other as the

Figure 89.

Figure 90. Multiple arch spans on solid rock footings. (This type of construction can nearly compete in first cost with steel construction, if the rock is exposed or covered with only a very shallow overburden.) In this case the rock in the stream is exposed at the pier locations during low water periods.

filled spandrel type. In the open spandrel type, the arch ribs (or barrel) support a series of columns which in turn carry the roadway deck. In the filled spandrel type, the arch barrel supports two longitudinal spandrel walls, one at either side of the roadway, which walls in turn retain a filling of earth or gravel upon which the roadway surface is placed.

Arch construction is further subdivided as follows:

(1) Rib arches (consisting of two or more independent arch units).

(2) Barrel arches wherein the individual ribs are replaced by one solid arch barrel.

A large portion of the cost in arch construction lies in the massive abutments and piers necessary to take the thrust of the arch ribs. For this reason the arch bridge finds special adaptation in the spanning of bold, rocky gorges, such as shown in Figure 88. In this figure, it will be noted that the arch ribs spring from bluff to bluff and that the rib thrust is thrown directly into the natural rock footing. Figure 89 illustrates another design with practically the same natural foundation conditions. Where natural rock is exposed throughout the bed of the stream, a series of multiple span arches may be constructed at an expense which, in general, will be very little greater than that for the construction of steel spans. Figure 90 is a series of multiple arch spans coming under this classification.

Arch spans are many times combined with reinforced concrete deck

Figure 91. Open spandrel arch construction in combination with deck girder approach spans carrying curved curtain walls.

Figure 92. A combination of arch and girder spans for a combined bridge and railroad overcrossing.

girder approaches carrying arched or curved curtain walls, thus giving continuity to the architectural treatment. Such an arrangement is shown in Figure 91. Figure 92 illustrates an arrangement of arch and girder spans for a combined railroad and river crossing. Typical details for the open spandrel arch shown in Figure 90 are included herewith. See Figures 93, 94, 95, 96 and 97. These may be of value in illustrating the general type of construction for which the curves have been prepared.

In the filled spandrel type of construction, the arch barrel supports two longitudinal retaining walls, parallel to, and at either side of the roadway, the space between these walls being filled with earth or gravel and the pavement surface being laid directly thereon. Figure 98 shows an arch structure of this kind.

Rib arches are also designed with curtain walls hiding the spandrel posts or columns so that the appearance presented is that of a filled spandrel arch. Figure 99 shows a reinforced concrete arch of this type with a stone veneered curtain wall.

Vertical loads on arch bridges operate to induce rib stresses which have a *horizontal* as well as a *vertical* component. Under yielding foundations, therefore, there is a tendency for the arch to spread at the footings, inducing stresses throughout the rib similar in direction to those induced by a drop in temperature, but much greater in magnitude. In many cases this has operated to induce stresses of such magnitude as to cause serious cracking throughout the rib. Figures 100 and 101 illustrate the effect upon arch superstructures of a yielding of the foundations. In these cases the foundation movement was the result of an undercutting

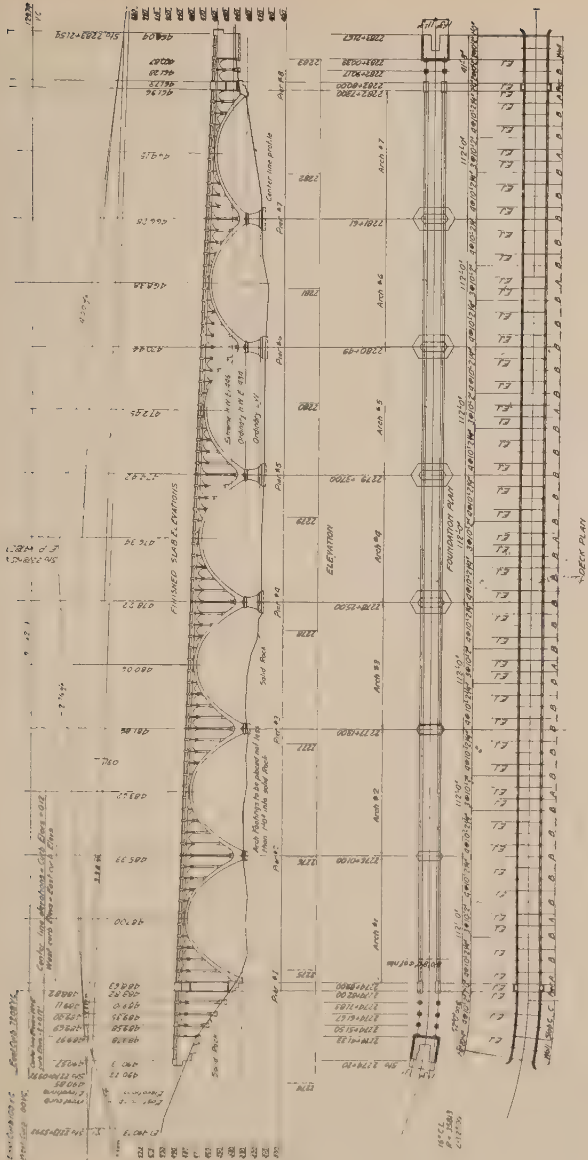


Figure 93. Plan, 7 112-foot R. C. arches with R. C. approaches.

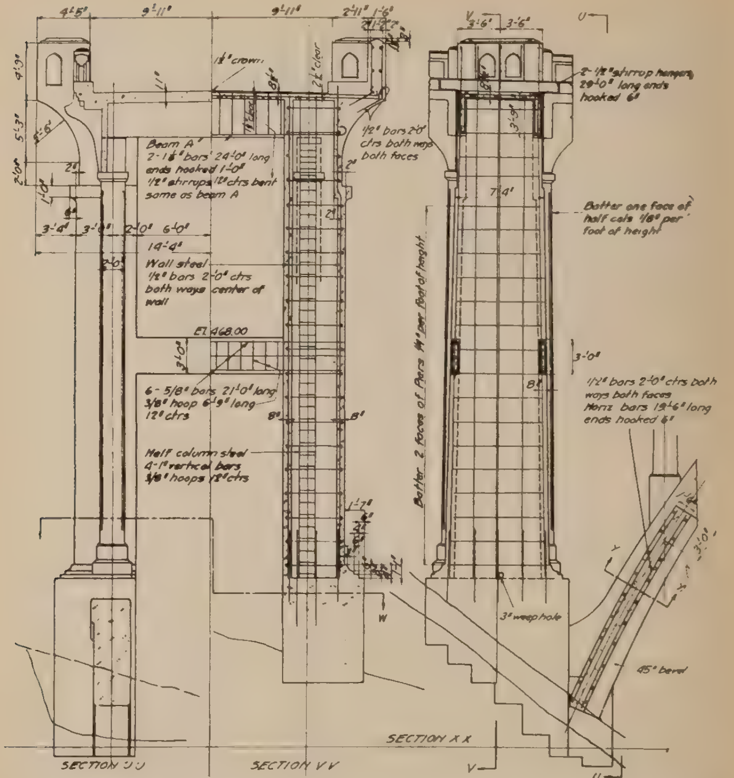


Figure 94. Cross section details, 7 112-foot R. C. arches with R. C. approaches.

and due directly to a failure to provide sufficient depth and mass for the abutments. Where solid foundations are not encountered at a reasonable depth, considerable expense must be incurred in producing the required fixity at the abutments. This is generally accomplished by constructing heavy mass abutments carried to a sufficient depth to insure the requisite immobility.

Arch abutments have been successfully founded upon piling in many instances. In this connection, however, it should be pointed out that the moving live loads operate to change the value of the lateral thrust components, inducing a churning action as the live load passes over the structure. Unless the design can be so arranged as to eliminate this

Figure 101. Failure of a fixed concrete arch bridge due to yielding of foundations.

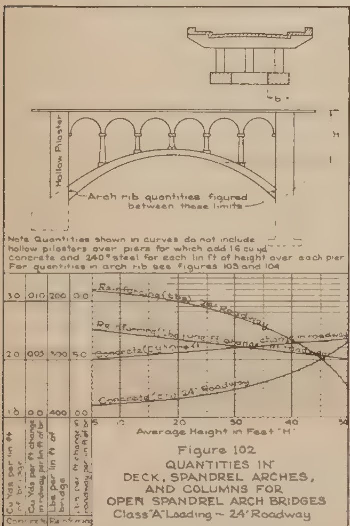


Figure 102
QUANTITIES IN DECK, SPANDREL ARCHES, AND COLUMNS FOR OPEN SPANDREL ARCH BRIDGES
Class "A" Loading ~ 24' Roadway

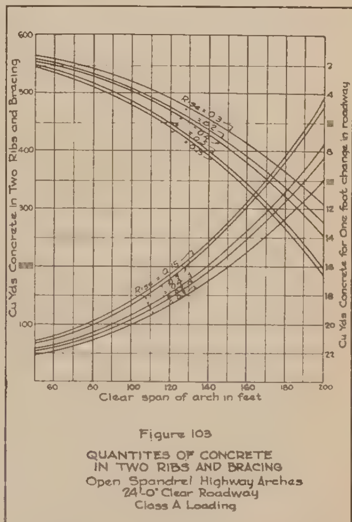


Figure 103
QUANTITES OF CONCRETE IN TWO RIBS AND BRACING
Open Spandrel Highway Arches
24'-0" Clear Roadway
Class A Loading

multiple span construction, where natural foundations are such as to reduce the cost of intermediate pier footings.

It is obviously impossible to develop quantity curves for this type which will fit every case, in view of the wide variation in the quantities

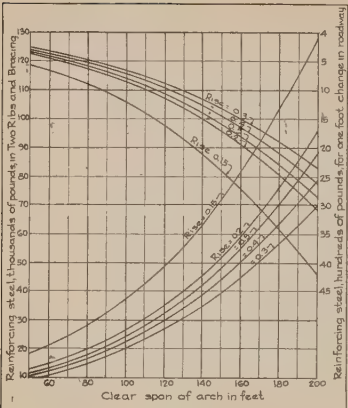


Figure 104

QUANTITIES OF REINFORCING IN TWO RIBS AND BRACING
Open Spandrel Highway Arches
24'-0" Clear Roadway
Class "A" Loading

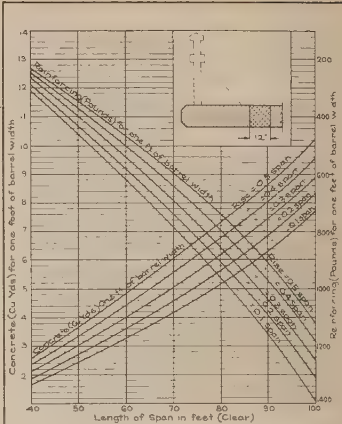


Figure 105

QUANTITY CURVES
FOR
BARREL ARCHES
WITH
FILLED SPANDRELS

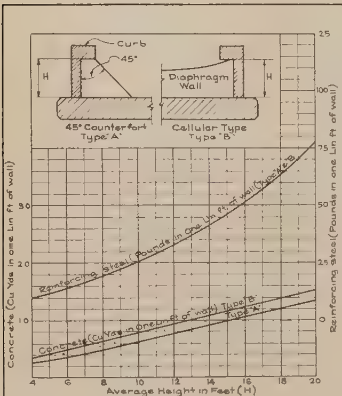


Figure 106

QUANTITY CURVES
FOR
SPANDREL WALLS
ON
FILLED SPANDREL ARCH BRIDGES

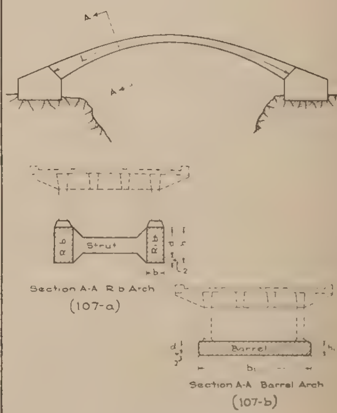


Figure 107

of material necessary for various arrangements of spandrel columns and various methods of architectural treatment. In Figures 102, 103 and 104 an attempt has been made to plot the quantities of concrete and reinforcing metal for arch structures of the rib type in span lengths varying from 60 feet to 200 feet with varying ratios of rise to span. These quantities must be regarded as a very rough blanket approximation, suitable only for the purpose of making preliminary estimates.

Figure 102 furnishes data for estimating the quantities of material needed for deck, spandrel arches, columns and pilasters for varying average heights "H," as shown. Figures 103 and 104 furnish similar data for the material in the arch ribs proper, Figure 103 being the data for concrete quantities and Figure 104 giving the approximate weights of reinforcing metal. Three rise to span ratios are plotted; intermediate values may be obtained by interpolation. To the quantities obtained from the above figures must be added the material necessary for abutments and piers and that needed for the handrailing. The abutment and pier quantities must be obtained in each individual instance as these will vary greatly with individual conditions. In general it is possible to prepare a reasonably close preliminary estimate by roughly sketching in the probable outlines of piers and abutments, and calculating the quantities therefrom. Handrail quantities may be obtained as noted hereinbelow for barrel arches.

In Figure 105 are given quantity curves for barrel arches with filled spandrels in span lengths ranging from 40 to 100 feet, and for varying rise to span ratios. These quantities are plotted from the average of a number of designs actually constructed and represent rough average values. To the quantities given in this last figure, must be added the quantities of material necessary for the spandrel walls and handrailings. Quantities for two types of spandrel walls are indicated in Figure 106, and quantities for the standard type of handrail shown in Figure 31 may be obtained from Figure 17. It will be noted that the range of span lengths used in the curve for filled spandrel arches is much less than for open spandrel rib arches. The reason for this lies in the fact that it is probably not true economy to construct filled spandrel arches in span lengths greater than 90 to 100 feet, owing to the uncertainty which exists as to the actual distribution of pressure through the earth filling and to the excessive dead load which is introduced by the same. The open spandrel arch, with its fixed points of load concentration, has a much sounder scientific basis than the filled spandrel design. Moreover, the stresses are considerably less, owing to the elimination of the excessive dead load introduced by the spandrel fill, as above noted. Many times

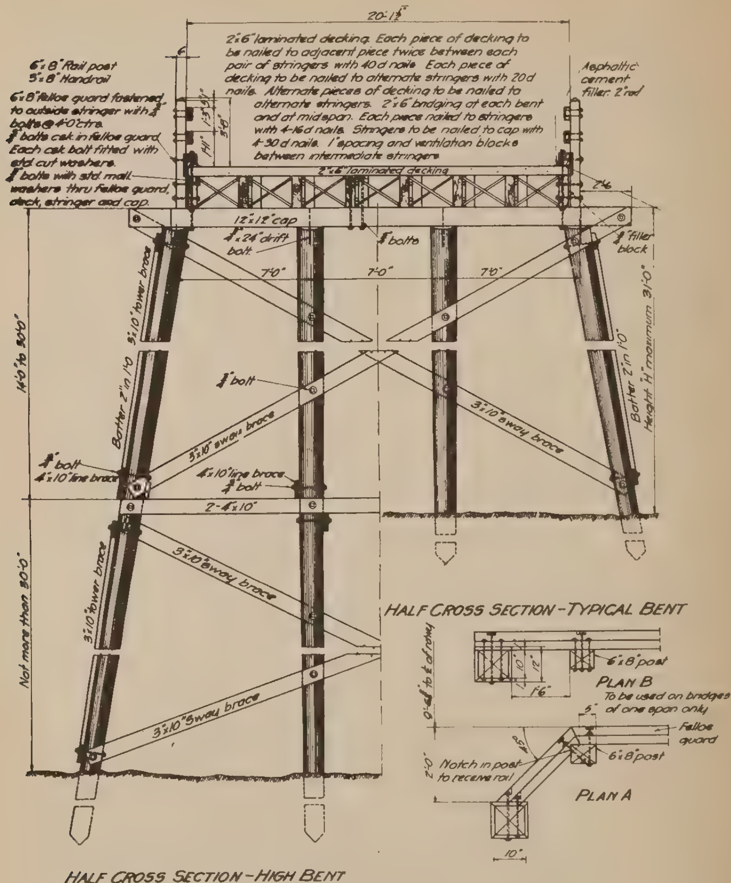


Figure 108. Typical pile trestle construction.

architectural considerations render it desirable to adopt the closed, or walled, spandrel for spans in excess of the above limit. When this is to be done, it is probably the part of wisdom to design the structure as an open spandrel arch, filling the spaces between the spandrel columns with solid curtain walls, as indicated in Figure 99.

The following table giving the quantities of material used in several typical rib arches recently constructed by the writer may be of value as an aid in preparing preliminary estimates.

QUANTITIES IN TYPICAL OPEN SPANDREL ARCH SPANS

Roadways 20 feet

Arch No.	Span	Rise	Number of Spans	Quantities per lin. ft. Con.	Quantities per lin. ft. Steel	Quantities per sq. ft. Con.	Quantities per sq. ft. Steel	Proportion of Quantities in preceding columns required to build concrete viaduct over same opening. Concrete	Steel
1	130	46	3	3.10	369	0.155	18.4	0.89	1.30
2	130	46	1	4.76	531	0.288	26.5	0.62	0.96
3	75	22	1	3.80	513	0.190	25.7	0.61	0.81
4	175	38	1	3.54	567	0.177	28.3	0.88	1.00
5	194	39	1	3.46	553	0.173	27.6	0.90	1.03
6	150	51	1	3.56	535	0.178	26.7	0.87	1.05
7	120	21	1	2.14	349	0.107	17.5	1.03	1.14
8	80	30	2	3.14	546	0.157	27.3	0.74	0.76
9	80	20	1	2.25	313	0.113	15.6	1.03	1.31
10	110	30.5	3	2.78	437	0.139	21.8	0.81	0.92
11	135	41	1	3.31	526	0.165	26.3	0.75	0.89
12	110	32	1	3.92	627	0.196	31.4	0.61	0.68
13	112	39 Av.	7	3.76	404	0.188	20.2	0.69	1.14

Note: The above quantities do not include piers and abutments below springing line.

It is sometimes considered advisable to adopt a solid barrel type of arch in preference to the rib design, even for open spandrel construction, owing to its greater rigidity under lateral impacts from drift or ice, or for other reasons. When this is done, an increase in quantities will result as indicated by the following discussion.

Consider the arch span shown in Figure 107a, the same consisting of two individual ribs of width b. Assume that the volume V for the two ribs is known from the curves shown in Figures 103 and 104, but that it is desired to ascertain the volume of a solid barrel arch such as shown at 107b, which will carry the same bending moment as the two individual ribs.

The dimension b_1 representing the width of the barrel arch, is assumed to fit the spacing of spandrel columns desired and to furnish the necessary lateral stability. It remains, therefore, to determine the relationship between the new depth h_1 at any point along the arch axis and the corresponding original rib depth h.

To accomplish this determination with any degree of precision is a rather difficult matter. The following approximate determination, however, is probably sufficiently exact for preliminary estimating.

Disregard all stresses resulting from temperature and rib shortening for the present and let M represent the bending moment at any section of the rib arch caused by dead load, live load and impact. Let it be further assumed that the variation in dead loading due to the adoption of the solid barrel arch type is so small that to all intents and purposes the bend-

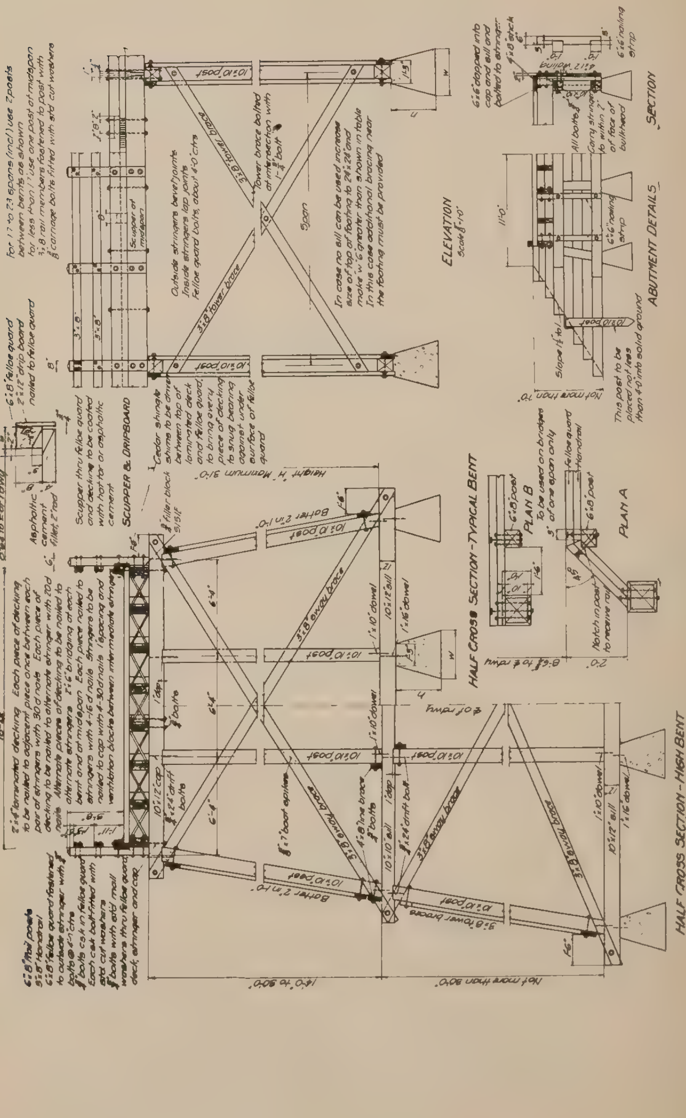


Figure 109. Typical frame trestle construction.

ing moment M_1 at the corresponding section is the same. In other words, let it be assumed that

$$M = M_1 \qquad \text{.......... 1}$$

Now

$$M = \frac{2f_c bd^2}{6} \text{ and } M_1 = \frac{f_c b_1 d_1^2}{6} \qquad \text{.......... 2}$$

Whence, for equivalent unit stress in the concrete

$$d_1 = \left[\sqrt{\frac{2b}{b_1}}\right] d \qquad \text{.......... 3}$$

The volume of the two individual ribs is represented by the expression:

$$V = 2Lb\ (d + 2'') \qquad \text{.......... 4}$$

Similarly

$$V_1 = Lb_1\ (d_1 + 2'') = Lb_1 \left[d \sqrt{\frac{2b}{b_1}}\right] + 2'' \qquad \text{.......... 5}$$

or approximately

$$Lb_1 \sqrt{\frac{2b}{b_1}}\ (d + 2'') \qquad \text{.......... 6}$$

Whence,

$$\frac{V_1}{V} = \frac{b_1}{2b} \frac{\sqrt{2b}}{\sqrt{b_1}} = \sqrt{\frac{b_1}{2b}} \qquad \text{.......... 7}$$

From the above expression the approximate volume of a solid barrel arch having a resisting moment equal to a rib arch of known dimension, may be readily calculated. For example, if the width of the barrel arch is to be taken as four times that of each rib of the rib arch, then the volume of concrete in the barrel arch necessary for an equivalent resisting moment is equal to the volume of the two ribs multiplied by $\sqrt{2}$ or 1.41.

The weight of reinforcing metal in the barrel arch span is readily computed from the following considerations:

$$A_s = \frac{M}{\frac{7}{8} f_s d} \qquad \text{.......... 8}$$

$$A_{s_1} = \frac{M_1}{\frac{7}{8} f_s d_1} \qquad \text{.......... 9}$$

Whence,

$$\frac{A_s}{A_{s_1}}=\frac{d_1}{d}=\sqrt{\frac{2b}{b_1}} \qquad \textbf{10}$$

or, expressed in words, the total weight of steel is inversely proportional to the rib depths at any point, or directly proportional to the square roots of the widths.

Thus far no mention has been made of temperature stresses or those resulting from axial thrust or "rib shortening." It can be easily demonstrated that the unit stress in the concrete resulting from temperature and rib shortening effects is a direct linear function of the rib or barrel depth. The actual temperature stresses in the barrel arch, therefore, will be less than in the rib arch by an amount given by the reduction ratio $\frac{h_1}{h}$ or approximately $\frac{d_1}{d}$. The total volume of concrete needed for the barrel arch may therefore be somewhat reduced from the values given by Equation 7, applied to the quantities given by the curves for rib arches. This volume must also be further reduced by the volume of concrete and weight of reinforcing metal in the struts between the individual arch ribs, as these are not used in the barrel arch type. Altogether this method of determination is rather crude, and approximate since the pro rata of stress caused by temperature and rib shortening must be assumed in order to apply the above correction. For rough preliminary estimates, a reduction coefficient of 85% may be applied to cover temperature effects and to allow for the volume of struts and braces. While this is rather inexact, the results are probably as close as can be obtained from the curves.

In this connection, it should be emphasized that the above method from start to finish is simply a means of arriving at a general idea of the quantities involved in any given design. Wherever the choice between types involves rather close decisions, the analysis should be carried further and detailed stress calculations made to determine the actual quantities required with a considerably greater degree of exactitude.

SECTION III—SUBSTRUCTURES

Article 17.—Pile Bents

Figures 3 and 108 illustrate typical pile bent construction, Figure 3 being for Class B loading, and Figure 108 being for Class A loading.

Article 18.—Frame Bents

Figures 2 and 109 are typical frame bent substructure designs, Figure 2 being for Class A loading, and Figure 109 being for Class B loading.

The choice between the frame, as against the pile type of bent is largely a matter of local conditions. Where foundations are of relatively soft material, pile bents are generally preferable. This type of substructure eliminates the danger of scour under any ordinary conditions (extreme flood conditions have been known to scour clear to the base of comparatively short piling, but such occurrences are rare). Frame construction, on the other hand, presents the advantage of being less easily deteriorated from decay. Frame bents on concrete pedestals may generally be set sufficiently high to maintain the majority of the lower sills in the dry, and even if the sills do eventually show indication of decay, it is not a difficult matter to remove and replace individual pieces under service. Piling, on the other hand, pass into the ground and are quite apt to show a comparatively rapid rate of decay at or near the ground line. Repairs in construction of this kind are more expensive and more difficult to make under traffic than in the case of frame construction. Where foundation conditions are such as to afford sufficient bearing for concrete pedestals, and the stream conditions are such as to eliminate any undue danger of erosion underneath the pedestals, frame construction presents distinct advantages. Mud sills, as a support for the sills of frame bents, are often used as a temporary expedient, but these result in rather heavy maintenance costs. It is sometimes considered the part of wisdom to adopt a pile bent for the first service period with a view to cutting off the pile below the ground line at such time as decay has developed sufficiently to render it of no further service and to use that portion of the pile remaining in the ground as a foundation upon which to rest a concrete pedestal and a frame bent.

Figure 110 contains a series of quantity curves for one bent of standard trestle substructure both pile and frame. The quantities for the frame construction do not include the concrete pedestals and those for the pile construction do not include the piling. These items must, therefore, be added to determine the total cost of the bent.

Figures 111a, 111b, 111c, and 112a, 112b, and 112c are *cost* curves corresponding to the quantities given in Figure 110 at varying unit prices.

Figures 113a to 113f inclusive, contain data as to quantities of concrete in square pedestals of varying top dimension, height and side batter. These curves are self-explanatory.

Article 19.—Light Concrete Pedestal Piers and Abutments

The term "light concrete pedestal piers and abutments" is used to refer to a rather inexpensive mass concrete construction used principally for supporting timber decks and other short spans. The design types for substructures of this class differ so widely in the different states, that

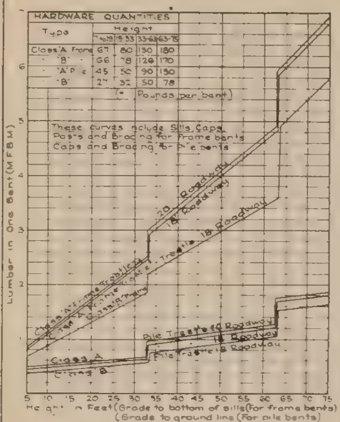


Figure 110
QUANTITY CURVES FOR
ONE BENT
STANDARD TRESTLE SUBSTRUCTURE

Note The above quantities do not include piling for pile trestle substructure

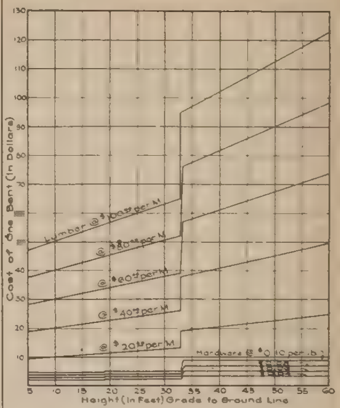


Figure 111a
COST CURVES FOR
STANDARD PILE TRESTLE BRIDGES
Substructure Only
Class A Loading 20' Roadway

Note Above amounts include tower bracing but do not include piling

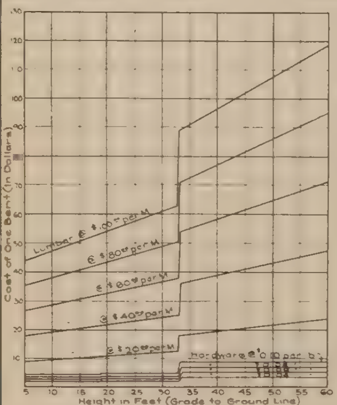


Figure 111b
COST CURVES FOR
STANDARD PILE TRESTLE BRIDGES
Substructure Only
Class A Loading 18' Roadway

Note Above amounts include tower bracing but do not include piling

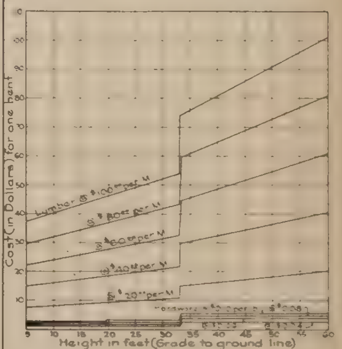


Figure 111c
COST CURVES FOR
STANDARD PILE TRESTLE BRIDGES
SUBSTRUCTURE ONLY
Class B Loading 18' Roadway

Note - Above amounts include tower bracing but do not include piling

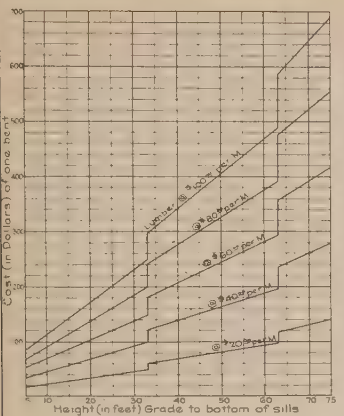


Figure 112a
COST CURVES FOR
STANDARD FRAME TRESTLE BRIDGES
SUBSTRUCTURE ONLY
Class "A" Loading 20' Roadway

Note - The above amounts include hardware erection, and creosote treatment For amount of hardware necessary see figure no 110

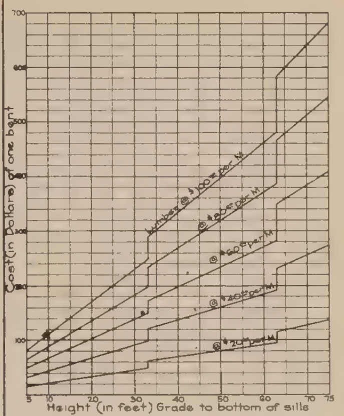


Figure 112b
COST CURVES FOR
STANDARD FRAME TRESTLE BRIDGES
SUBSTRUCTURE ONLY
Class "A" Loading 18' Roadway

Note:- The above amounts include hardware, erection, and creosote treatment For amount of hardware necessary see figure no 110

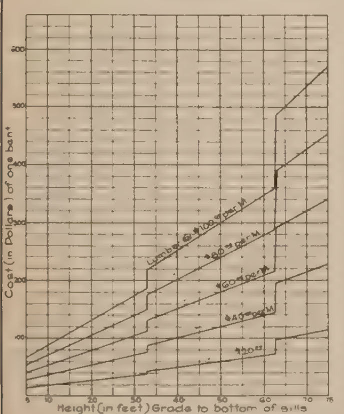


Figure 112c
COST CURVES FOR
STANDARD FRAME TRESTLE BRIDGES
SUBSTRUCTURE ONLY
Class "B" Loading 18' Roadway

Note:- The above amounts include hardware, erection and creosote treatment For amount of hardware necessary see figure no 110

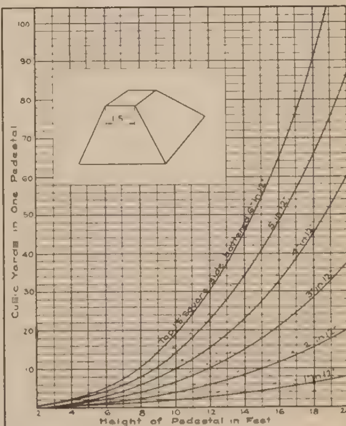


Figure 113a
CURVES SHOWING
QUANTITIES IN SQUARE PEDESTALS
WITH
SIDES BATTERED DIFFERENT AMOUNTS
TOP 15' SQUARE

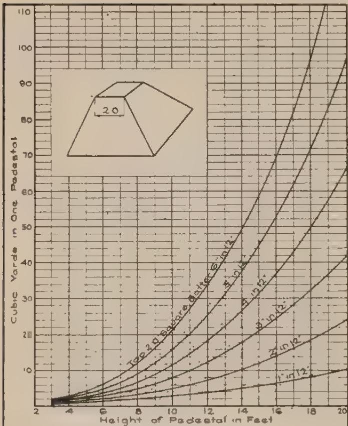


Figure 113 b

CURVES SHOWING
QUANTITIES IN SQUARE PEDESTALS
WITH
SIDES BATTERED DIFFERENT AMOUNTS
TOP 2.0 SQUARE

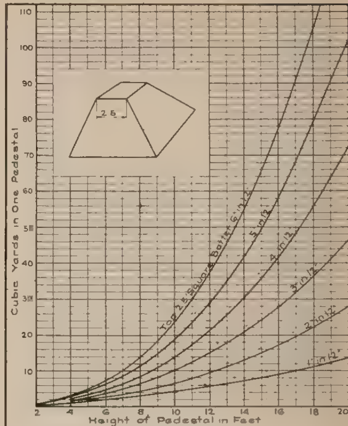


Figure 113 c

CURVES SHOWING
QUANTITIES IN SQUARE PEDESTALS
WITH
SIDES BATTERED DIFFERENT AMOUNTS
TOP 2.5' SQUARE

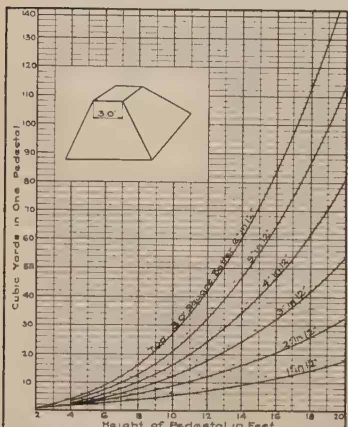


Figure 113 d

CURVES SHOWING
QUANTITIES IN SQUARE PEDESTALS
WITH
SIDES BATTERED DIFFERENT AMOUNTS
TOP 3.0 SQUARE

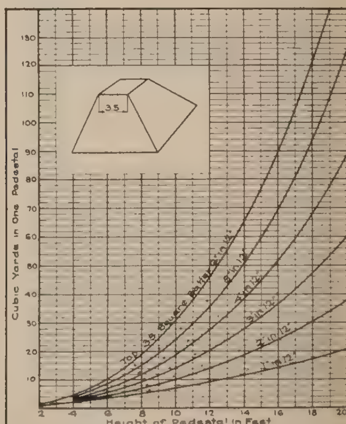


Figure 113 e

CURVES SHOWING
QUANTITIES IN SQUARE PEDESTALS
WITH
SIDES BATTERED DIFFERENT AMOUNTS
TOP 3.5' SQUARE

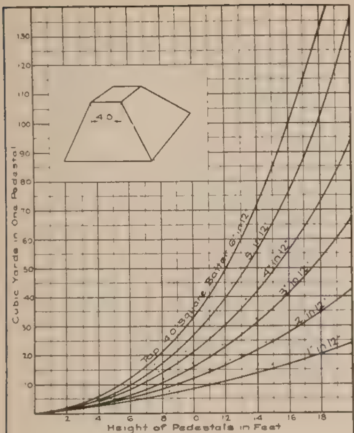


Figure 113f
CURVES SHOWING QUANTITIES IN SQUARE PEDESTALS WITH SIDES BATTERED DIFFERENT AMOUNTS TOP 4'0' SQUARE

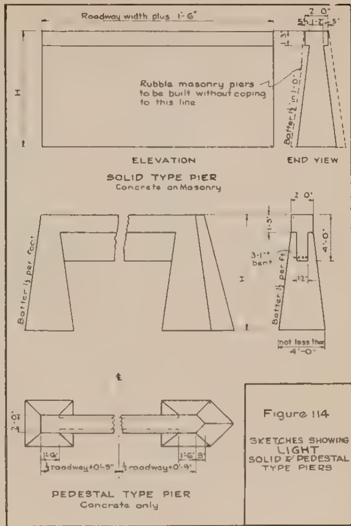


Figure 114
SKETCHES SHOWING LIGHT SOLID & PEDESTAL TYPE PIERS

it is impossible to compute quantity curves which will represent a uniform standard practice. Figure 114 indicates two types of construction commonly employed; the pedestal type, shown in the lower half of the figure, consists of two independent pedestals joined by a connecting strut or beam at the top. This type of substructure finds a useful field of application for the support of timber A-frames and even for longer spans where the superimposed load is not too great for the pedestals to carry, and foundation conditions are such as to eliminate the possibility of erosive tendencies. The upper portion of Figure 114 indicates a typical design for a solid type of light pier employed for concrete or rubble masonry construction. It has seemed the part of wisdom in construction of this kind, to design with rather more liberal dimensioning in the case of rubble masonry than for concrete construction, as shown by the dotted lines in the above figure, and the quantity curves have been computed on this basis. Figure 115 indicates the quantities of material required for each of the above described types of construction for a 20-foot roadway bridge, and also the yardage needed for each foot additional roadway width. Figure 116 is a photographic view of a design utilizing this type of construction in connection with wooden trestle decks. In certain cases it may be considered advisable to use a type of construction similar to the solid pier shown in Figure 114, as an abutment without wings, the ap-

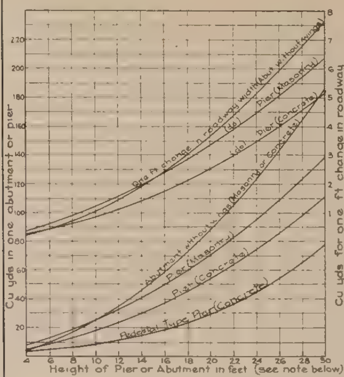


Figure 115

CURVES SHOWING QUANTITIES
IN
MASS CONCRETE AND MASONRY
ABUTMENTS AND PIERS
20' Roadway

Note Height of piers measured from bridge seat to base of footing, abutments from grade to base of footing

Figure 116. Solid light type concrete piers supporting timber trestle deck.

proach fill in such a case extending around the end of the abutment and, therefore, obstructing a portion of the waterway. Considerable saving can be affected by dispensing with wing walls as above described, where hydraulic conditions permit; it is sometimes necessary, however, to pave the slopes with concrete blocks or hand-placed stone riprap in order to avoid erosion at the toe of the fill. Even with this added expense, however, the saving over the cost of wing walls is ofttimes quite marked.

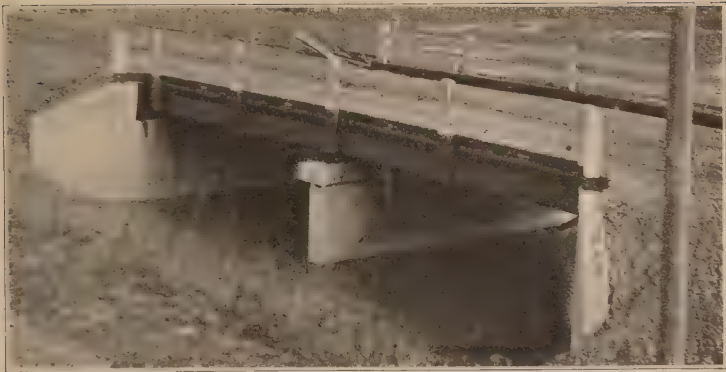


Figure 117. Reinforced concrete abutment and piers, carrying a temporary deck.

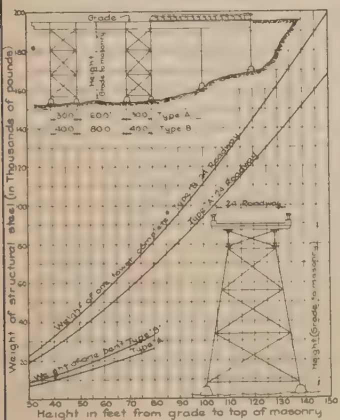


Figure 118
QUANTITY CURVES
SHOWING AMOUNT OF STRUCTURAL STEEL
IN STEEL VIADUCT TOWERS
AND BENTS

*Weight of towers does not include weight of girders

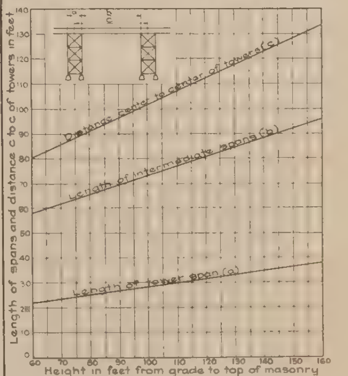


Figure 119
CURVES SHOWING
APPROXIMATE ECONOMIC SPAN LENGTHS
FOR STEEL VIADUCTS
OF
VARIOUS HEIGHTS

Note
Calculated for 24 roadway, concrete deck & railing

When this is done, an abutment section similar to that shown in Figure 125 is employed. Quantities for abutment piers of this type are also indicated in Figure 114.

The objection to the pedestal type of pier indicated in Figure 114 is the danger of damage from drift and ice, or any other like condition which militates against light construction of this type. The first cost of the

pedestal type, however, is considerably less than the other and where conditions warrant its use, a considerable saving can be affected thereby.

A great variety of abutment and pier types may be developed from the above standard types. For example, the pedestal type may be utilized with a solid web or diaphragm from top to ground line, or, again, the pedestal type may be modified by placing both pedestals upon an integral footing, bracing between the individual pedestals by means of struts or a continuous diaphragm. The quantities shown in Figure 114, however, represent about the extremes of practice in this regard, any modification in design generally falling somewhere midway between the extreme limiting curves shown. These light types of abutment and pier design must not be confused with standard reinforced concrete abutments or with the standard heavy type piers used for supporting heavier construction types, which types will be described later on. The principal field of utility for this type of construction is for supporting short span superstructures under conditions which do not require the use of a heavier type of substructure construction. In many cases, it is considered advisable to adopt standard reinforced concrete abutments and piers, as described later on (see Figures 132 to 138), in combination with a temporary timber superstructure to be replaced at the end of its economic service period by a superstructure of reinforced concrete. Figure 117 is a structure of this kind, the piers and abutments being standard construction and carried well below stream bed, and set on wide spread footings. The substructure in this case was designed for true permanence and proportioned to carry a future reinforced concrete slab deck and railing.

Article 20.—Steel Viaduct Towers

Figure 118 indicates the approximate weight of structural steel in one tower or one bent for two types of standard viaduct construction, designed for a reinforced concrete deck and rail and a 24-foot roadway. The quantities represented in this figure are somewhat in excess of those ordinarily used, as a rather heavy and rigid type of tower has been employed. It seems the part of wisdom to go to this expense for construction of this type in view of the fact that its use can only be justified in view of a very long service life. This type of structure is in price competition with the reinforced concrete viaduct type described later on (see Article 21) for heights up to 60 feet (grade to base of footing). Above this point it is in competition with long spans and high piers of various and sundry types. For heights below 75 or 80 feet, frame trestle construction may also be considered as an alternate solution in temporary construction, unless waterway or other considerations render it necessary to adopt a span length in excess of 20 to 24 feet, in which case the frame

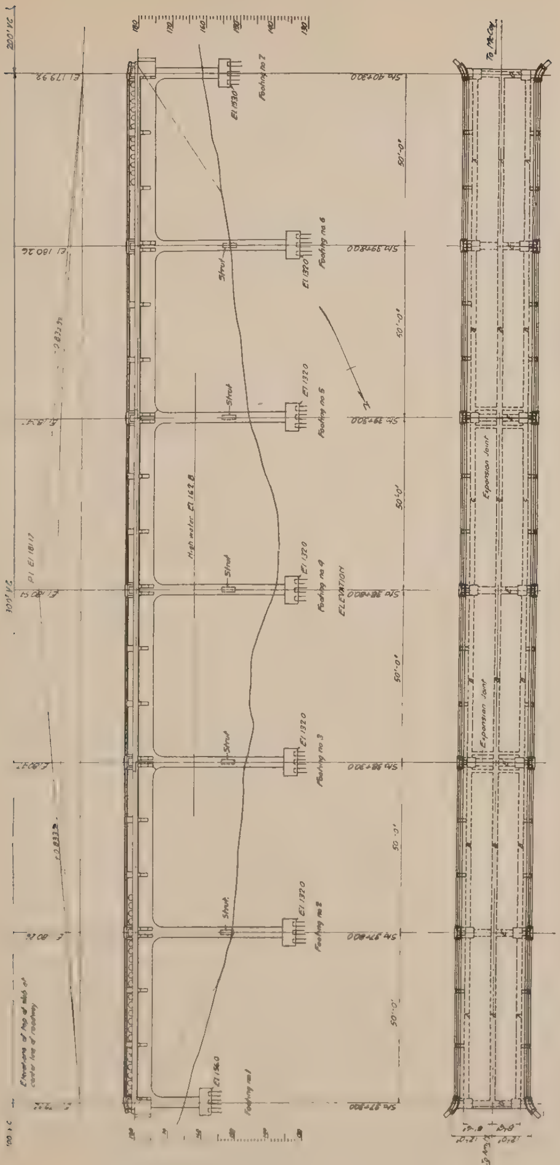


Figure 120. General layout, typical R. C. viaduct construction.

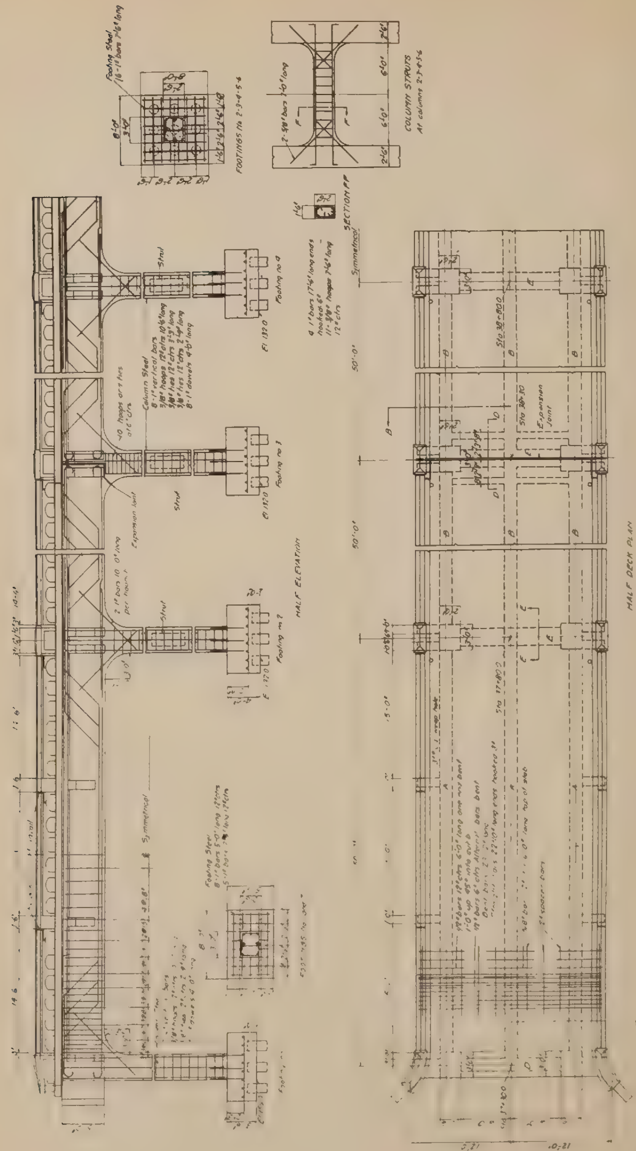


Figure 121. Details, typical R. C. viaduct construction.

trestle is automatically out of the race.

On Figure 119 is plotted a series of curves showing the approximate economic span lengths for steel viaducts of various heights. This curve is adapted from a similar study made of railroad viaducts by Dr. J. A. L. Waddell and given in his treatise on bridge engineering.

It would be possible to adopt a type of tower construction somewhat lighter than the types from which the above curves have been plotted, although the propriety of such a procedure is somewhat doubtful. At the most, a saving of from fifteen to twenty percent in metal would be affected.

It will be noted that the above curves do not include the metal necessary for superstructures. The superstructure design will consist merely of deck plate girders, or trusses, and quantities for the same may be obtained from Figures 83, 86 or 87.

Article 21.—Reinforced Concrete Viaduct Construction

This type of construction consists in a series of reinforced concrete beam or girder superstructures, supported upon concrete columns, sometimes braced in the form of towers. Figure 120 is a typical general drawing of viaduct construction of this type. Details for this type of construction are shown in Figure 121. Figure 122 contains quantity curves for concrete viaduct towers for a 24-foot roadway, exclusive of the superstructure, and also the quantities involved for each foot change in roadway width. Quantities are given both for the two post, and the three post bent types. It will be noted that a footing depth of 2 feet has been assumed in every case. Where piling are to be used, this depth will not be sufficient, and in many other cases it will be found advantageous to increase footing depths. Where this is done, it will be necessary to increase the quantities shown in Figure 122 accordingly. Figure 123 is a photographic view of a triple span, two post, reinforced concrete viaduct, while Figure 124 shows a longer three post viaduct.

Article 22.—Heavy Gravity Abutments

Figure 125 indicates a typical design for mass or gravity abutments. Quantities for these are plotted in curve form in Figure 126. These quantities represent average practice in regard to design for substructures of this character and are doubtless self-explanatory. It will be noted that quantities are plotted both for stone masonry, and for concrete, the dimensions in the case of the stone masonry construction being taken more liberally, as indicated in Figure 125.

It is also noted that for each abutment height, quantities have been plotted for two wing wall lengths, representing about the extreme minimum

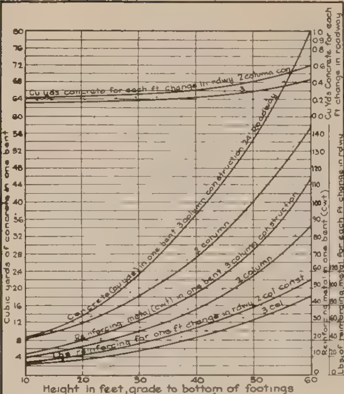


Figure 122
QUANTITY CURVES
FOR
CONCRETE VIADUCT TOWERS
24' Roadway

Note The above quantities include cross girders, all necessary bracing and footings 2 feet in thickness If additional material is added to footings, same must be added to quantities obtained from curves

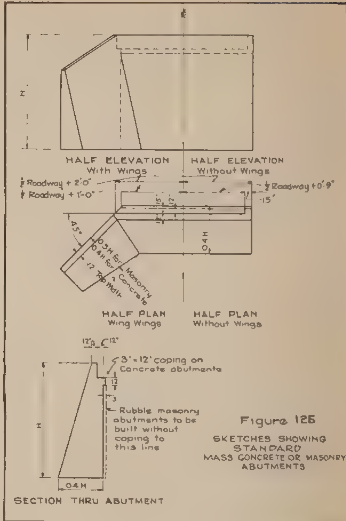


Figure 125
SKETCHES SHOWING
STANDARD
MASS CONCRETE OR MASONRY
ABUTMENTS

Figure 123. Typical two-post viaduct design.

Figure 124. Typical three-post reinforced concrete viaduct.

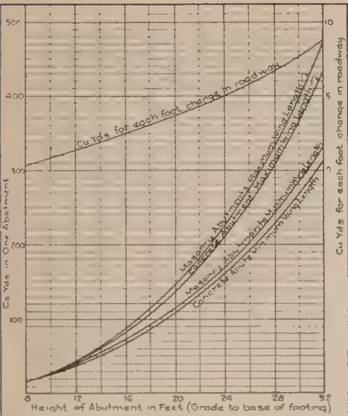
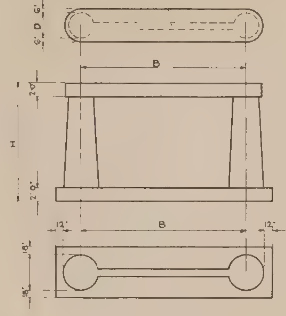


Figure 126

CURVES SHOWING QUANTITIES IN STANDARD MASS CONCRETE & MASONRY ABUTMENTS

20' Roadway



Note
Bases of shafts up to 24' high to be 10' larger in diameter than top
Above 24' high batter on shafts to be ½" per foot of height

Figure 127

SKETCH SHOWING DUMB-BELL PIERS

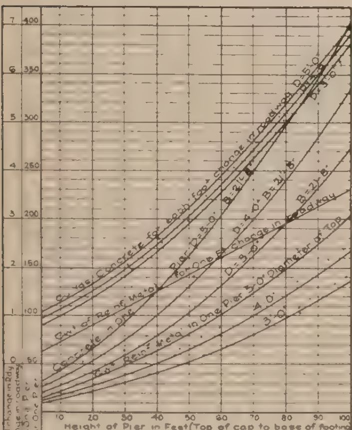


Figure 128

QUANTITY CURVES FOR STANDARD DUMB-BELL PIERS

21'-8" ¢ ¢ shafts

For dimensions see figure 127

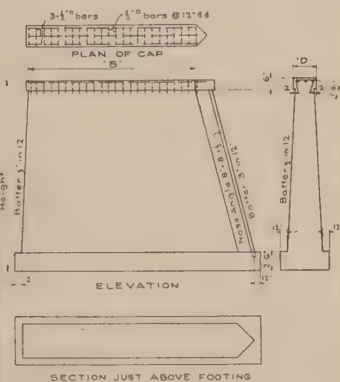


Figure 129

SKETCH SHOWING MASS TYPE CONCRETE PIERS

TRIANGULAR NOSE ON ONE END

and maximum likely to be encountered for right angle abutments. Quantities for wing lengths of intermediate value may be obtained, with a reasonable degree of exactitude, by interpolation.

Article 23.—Concrete Piers

Figure 127 illustrates a type of pier design known as the dumb-bell type, consisting of two battered circular pedestals connected by a continuous diaphragm, the entire pier being set on a heavy spread base. Figure 128 indicates the quantities of material in this type of pier for various heights and widths. Figure 129 is a sketch of another type of pier construction adopted in many of the states. This type contains considerably more material, being of a solid rectangular cross-section, with the necessary nosing and ice breakers. The quantities of material for this type of pier design for different heights and widths are shown in Figure 130.

The round nosings shown on the dumb-bell type of pier are generally to be preferred over the angular type of nosing, for which reason another type of pier similar to the last type above, but with rounded nosings, is sometimes employed, quantities for which are indicated in Figures 131a, b and c.

It will be observed that the quantity curves for the round nosed piers make no provision for the additional material needed for ice breakers. Detailed designs for ice-breakers of this class vary with the local requirements to such an extent as to render it rather difficult to plot any series of quantity curves which will fit every condition. A rough pencil sketch of the additional cross-section necessary may be made and the additional yardage required for the necessary ice-breakers readily computed and added to the quantities shown in the above figures.

Article 24.—Reinforced Concrete Abutments and Wing Walls

Abutments, for purposes of classification and quantity calculation, may be divided into two component parts, as follows:

1. Abutment body.
2. Wing walls.

There are two types of abutment body in common use, as follows:

1. Abutment body integral with superstructure.
2. Abutment body not integral with superstructure.

Taking these up in order, the abutment body integral with the superstructure is designed as shown in Figure 132. No expansion joints are provided at the junction between abutment and superstructure, the two component parts being joined together by a deep haunch connection, heavily reinforced, making the superstructure and substructure act as

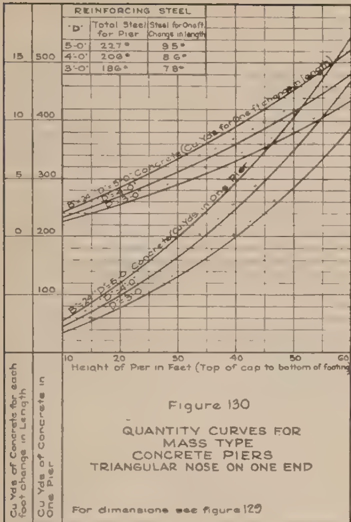


Figure 130

QUANTITY CURVES FOR MASS TYPE CONCRETE PIERS TRIANGULAR NOSE ON ONE END

For dimensions see figure 129

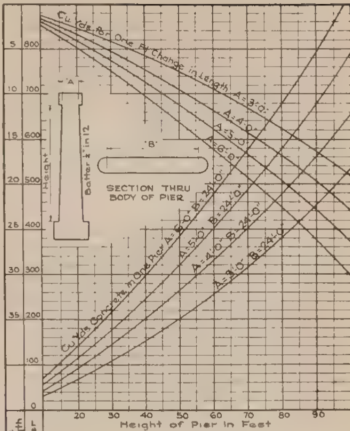


Figure 131 a

CUBICAL CONTENTS OF MASS PIERS WITH ROUNDED ENDS

Batter ¼" in 12"

Note The above quantities do not include copings or bases

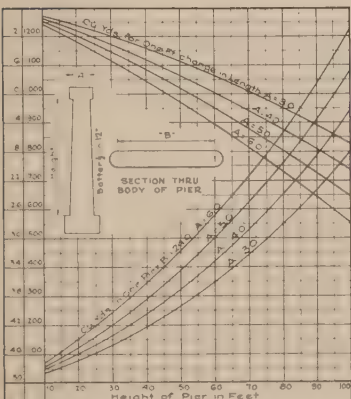


Figure 131 b

CUBICAL CONTENTS OF MASS PIERS WITH ROUNDED ENDS

Batter ½" in 12"

Note The above quantities do not include copings or bases

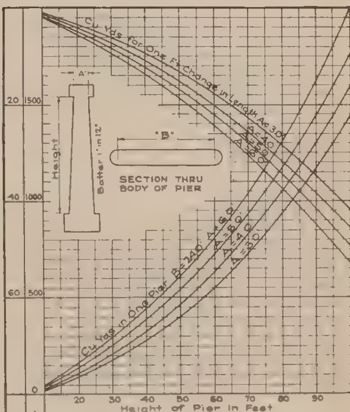
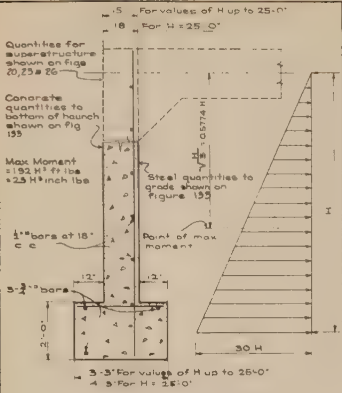


Figure 131 c

CUBICAL CONTENTS OF MASS PIERS WITH ROUNDED ENDS

Batter 1" in 12"

Note The above quantities do not include copings or bases



Note Quantities in figure 133 are computed for footing dimensions shown above if these are inadequate for vertical bearing quantities must be increased accordingly

Figure 132
ABUTMENT SECTION
FOR
ABUTMENTS INTEGRAL WITH SUPERSTRUCTURE

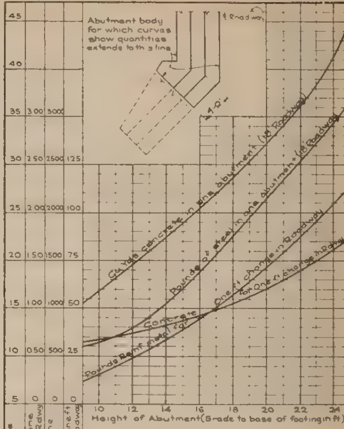


Figure 133
QUANTITY CURVES
FOR
CONCRETE ABUTMENT BODIES
ABUTMENT INTEGRAL WITH SUPERSTRUCTURE

Note - For Quantities in wings see figure 138 & 139

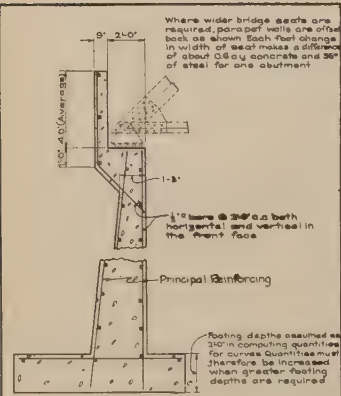


Figure 134
SKETCH SHOWING
CROSS SECTION THRU ABUTMENT
NOT INTEGRAL WITH SUPERSTRUCTURE

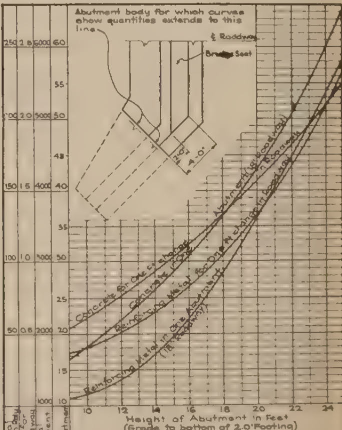


Figure 135
QUANTITY CURVES
FOR
CONCRETE ABUTMENT BODIES
ABUTMENT NOT INTEGRAL WITH SUPERSTRUCTURE

Note: For quantities in wings see Figures 138 & 139

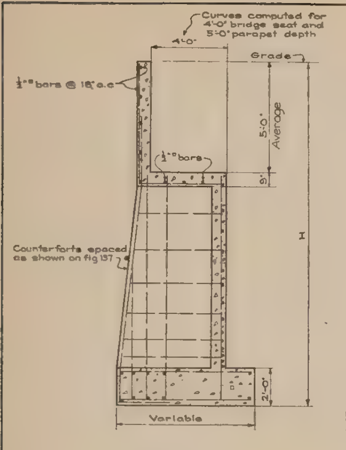


Figure 136
COUNTERFORT ABUTMENT
SECTION
Quantities shown on Figure 137

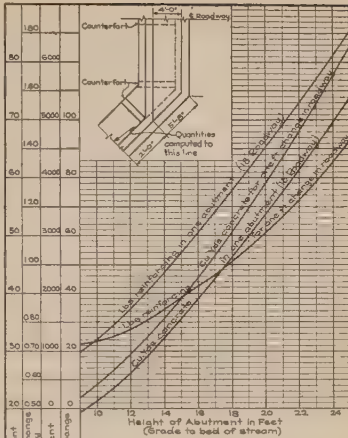


Figure 137
QUANTITY CURVES
FOR
COUNTERFORTED ABUTMENTS
WITH WIDE BRIDGE SEAT

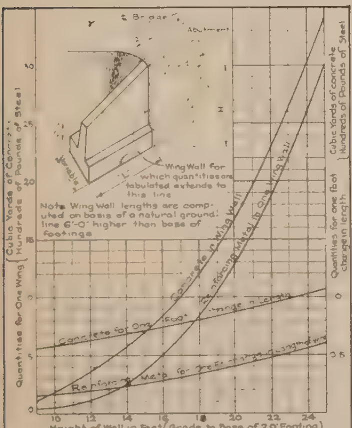


Figure 138
QUANTITY CURVES
FOR
CONCRETE WING WALLS
Vertically Cantilevered

Note The quantities shown above are for a 2'0" footing depth Where for any reason the footings are made deeper the quantities must be increased accordingly.

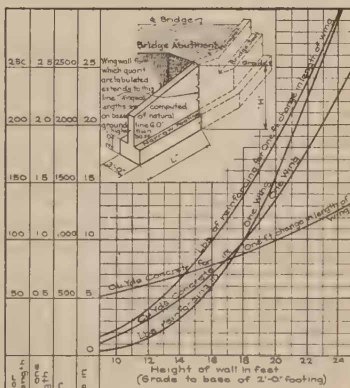


Figure 139
QUANTITY CURVES
FOR
HORIZONTALLY CANTILEVERED
WING WALLS

Note Quantities shown above are for a 2'0" footing depth and must be increased where for any reason footings are made deeper

a unit. It will be noted that in designs of this character, the abutment body acts as a simple beam, restrained at the base and at the top along the center of pressure of the superstructure. This vertical beam is under the action of the horizontal pressure induced by the restrained material of the approach fill in combination with the direct vertical pressure induced by the superimposed dead and live loadings. Figure 133 consists of quantity curves for concrete and reinforcing metal for abutments of this type in heights up to 25 feet.

Figure 134 indicates an abutment design wherein an expansion joint is provided between substructure and superstructure, rendering it necessary for the abutment body to act as a vertical cantilever under the horizontal pressure of the restrained filling material. The quantities of material necessary for abutment bodies designed in this manner are given in Figure 135.

Figure 136 illustrates a type of abutment body consisting of counterforts connected by a thin reinforced face wall. This type of design is especially adapted to substructures requiring a deep bridge seat. Quantities of material for this type of design are shown in Figure 137.

It will be noted that the integral superstructure operates to reduce the quantities in the abutment body. This type of design, therefore, finds a field of utility for short spans and for the end spans of multiple span construction wherever it is possible to dispense with an expansion joint. It is unquestionably true that this type of design induces rather heavy temperature stresses in the abutment stem and in the superstructure, which stresses operate to produce cracks at or near the haunches. Through the adoption of deep and heavily reinforced haunches, however, these temperature stresses can be taken care of so that the above type of substructure may be used for spans up to 40 feet in length without serious danger of cracking.

Wing walls may be divided into two classes, as follows:

1. Vertical cantilevers.
2. Horizontal cantilevers.

The vertical cantilevered type is indicated in Figure 138, which figure also includes a curve of quantities for wing walls of various heights. In this type of design, the wing wall is assumed to act as a vertical cantilever under the action of the restrained filling material. It will be noted that the wing wall lengths have been computed on the basis of a location of solid ground at the abutment face at an elevation 6 feet above the base of the footing, this being considered a rough average condition. If the ground profile varies from this assumption, or if the use of skew abutments, the presence of bends in the stream, or any other local condition renders it necessary to adopt longer wings (see Figure 140), the curve

quantities should be increased by the amounts given by the curves for "one foot change in length," multiplied, of course, by the number of feet by which the wings are lengthened.

In the horizontally cantilevered type of wing design (Figure 139), the footings are very narrow, their function being only to support the wing wall against settlement under its own weight, the wing stresses in this type being carried horizontally into the abutment body. Quantities for this type of wall for various heights are given in Figure 139.

In the above discussion, vertically cantilevered abutment bodies have been considered as acting independently of the attached wing walls, no consideration being given to the restraining action of the wing walls against overturning. In fact, it is doubtless better practice to cut the wing walls entirely loose from the abutment body in order to avoid the tendency toward crack formation at the junction. If, however, the wing walls and abutments are placed as a monolithic unit, a consideration of Figure 141 would indicate that such wing walls operate to shift the center of gravity of the abutment mass rearwardly and, therefore, operate to reduce the footing stresses. As the abutment cantilever tends to tip forward under earth pressure, it has an obvious tendency to lift the wing walls with it. These walls are restrained from movement by the friction of the earth along their inner faces, by their own weight and by the weight of the filling material superimposed upon their spread footings. A considerable retraining action therefore exists, but in order to render this restraint effective, a system of tension rods thoroughly and effectively tying the wing wall and abutment body into one monolithic mass must be employed. Figures 141 and 142 indicate a standard type of design, wherein tension rods are employed to utilize the restraining action above described.

Figures 143a and 143b indicate the dimensions and the quantities of concrete and reinforcing metal necessary for the above type of design for various abutment heights and widths of roadway, these quantities being taken from the standard abutment plans adopted by the Iowa State Highway Commission.

SECTION IV.—MISCELLANEOUS TYPES AND COST DATA

ARTICLE 25.—FILLED APPROACHES

Figures 144, 145 and 146 are curves showing the cost per lin. ft. of embankment for various fill heights and shoulder widths and for various yardage prices. Where the waterway conditions are such as to render it possible to place a filled embankment over a portion of the overflow flat, the matter of choice between fill versus structural approach becomes one

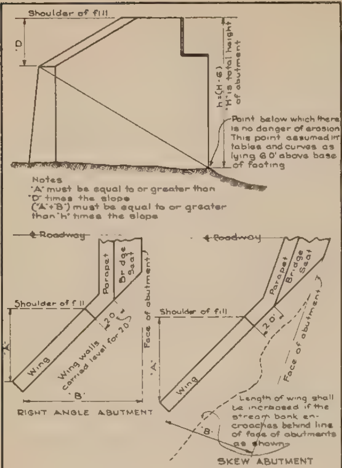


Figure 140
INDICATING METHOD BY WHICH WING WALLS GIVEN IN FIGURES 138-9 ARE PROPORTIONED FOR LENGTH

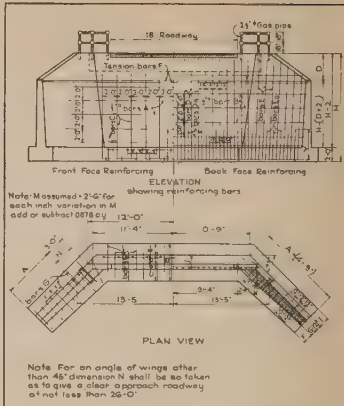


Figure 141
Monolithic Abutments with Tension Bars for wing anchorage. (For Steel Superstructure)

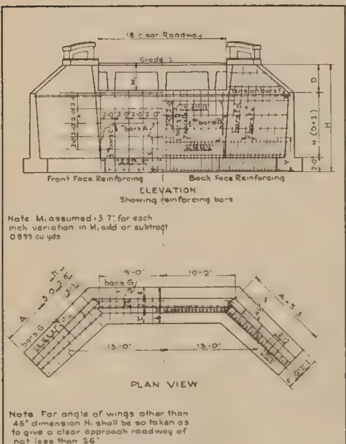


Figure 142
Monolithic Abutments with tension bars for wing anchorage (For Concrete Superstructure)

ABUTMENT DIMENSIONS	H=Height		10'	11	12'	13'	14'	15	16'	17'	18'
	A-Minimum		3-0"	3-6"	4-0"	5-0"	6-0"	7-0"	8-0"	9-3"	10-3"
	A Maximum		4-3	5-6"	6-6"	7-6"	8-6	9-9"	.0-9"	11-9"	12-9"
	B		3-4"	3-8"	4-0"	4-4"	4-8"	5-0"	5-4"	5-8"	6-0"
	C		8"	10"	12"	1-2"	1-4"	1-6"	1-8"	1-10"	2-0"
	D for min wing		1-3"	1-6"	1-9"	2-3"	2-9"	3-0"	3-6"	4-3"	4-9"
	D for max wing		2-0"	2-6"	3-0"	3-6"	4-0"	4-6"	5-0"	5-6"	6-0"
	E		8"	10"	12"	1-2"	1-4"	1-6"	1-8"	1-10"	2-0"
	F		1-0"	1-1"	1-2"	1-3	1-4"	1-6"	1-8"	1-10"	2-0"
	G for min wing		6¾	7½"	8¼"	8¾"	9¼"	10"	10½"	10¾"	11¼"
	G for max wing		6"	6½"	7"	7½"	8	8½"	9"	9½"	10"
	I		1-0"	1-1"	1-2"	1-3"	1-4	6"	1-8	1-10"	2-0"
	J		2-0"	2-0"	2-0"	2-0"	2-0	2-0"	2-0"	2-0"	2-0"
	Bars D		½"□	½"□	½"□	¾"□	¾"□	¾"□	¾"□	¾"□	¾"□
	Bars Da		none	none	none	none	none	none	¾"□	¾"□	¾"□
	Distance X								5-0	6-0	7-0"
	Bars E		½"□	½"□	½"□	¾"□	¾"□	¾"□	¾"□	¾"□	¾"□
	Bars Ea		none	none	none	none	none	none	¾"□	¾"□	¾"□
	Distance Y								5-0"	6-0"	7-0"
	Bar F		2-¾"□	2-¾"□	2-¾"□	2-¾"□	2-⅞"□	2-1"□	3-1"□	3-1⅛"□	4-1⅛"□
	Bar G		none	none	none	none	none	½"□	½"□	½"□	½"□
	Distance Z							12"	10"	8"	6"
QUANTITIES	With min wings 18' Roadway	Cu. Yds Conc	24.28	28.12	32.19	37.24	42.60	48.64	55.00	62.19	69.38
		Pounds Steel	744	800	9 0	1510	1746	2156	2915	3506	4162
	With max wing 18' Roadway	Conc	25.64	30.53	35.50	40.78	46.40	53.07	58.80	66.90	74.42
		Steel	779	868	005	684	930	2380	3290	3770	4556
	Add or subtract for each wing per ft change in length of wing	Conc	.544	.602	.662	.708	.760	.805	.875	.942	.008
		Steel	14.0	15.8	19.0	34.8	36.8	40.7	57.3	52.8	78.8
	Add or subtract per foot change in roadway	Conc	.703	.802	.900	1.000	1.098	1.197	1.296	1.394	1.493
		Steel	22.1	23.4	26.4	41.4	46.0	54.2	71.5	81.2	91.5

Figure 143a
TABLE A
Table of dimensions and quantities for design shown on figure 141

			10	11'	12'	13'	14	15'	16'	17'	18'
	H: Height		10	11'	12'	13'	14	15'	16'	17'	18'
ABUTMENT DIMENSIONS	A maximum		4 3'	5-6'	6-6	7-6'	8'-6'	9-9'	10-9	11-9'	12'-9'
	A minimum		3 0'	3 6	4-0	5 0'	6 0'	7-0'	8-0	9-3'	10-3'
	B		3-4'	3 8'	4-0'	4-4'	4 8'	5 0'	5-4'	5 8'	6-0'
	C		11"	11½"	12"	14"	16"	18"	20"	22"	24"
	D for max wing		2 0	2-6'	3-0	3 6'	4-0'	4-6'	5-0'	5-6'	6-0'
	D for min wing		1-3'	1-6	1-9'	2-3'	2-9'	3-0'	3-6'	4-3	4'-9'
	E		11"	11½	12"	14"	16"	18"	20"	22"	24"
	F		12"	13"	14"	15"	16	18"	20"	22"	24"
	G for max wing		6"	6½"	7"	7½"	8"	8½"	9"	9½"	10"
	G for min wing		6¾"	7½	8¼"	8¾"	9¼	10	10½"	10¾"	11¼"
	I		12"	13"	14"	15	16"	18"	20"	22	24"
	J		18	21"	24	24"	24"	24"	24"	24"	24"
	Bars D		½ □	½"□	½"□	¾"□	¾ □	¾ □	¾"□	¾"□	¾"□
	Bars Da		none	none	none	none	none	none	¾ □	¾"□	¾"□
	Distance X								5 0'	6'-0'	7-0'
	Bars E		½ □	½"□	½"□	¾ □	¾"□	¾"□	¾"□	¾ □	¾"□
	Bars Ea		none	none	none	none	none	none	¾"□	¾"□	¾"□
	Distance Y								5'-0'	6'-0'	7'-0'
	Bars F		2 ¾"□	2 ¾"□	2-¾"□	2-¾"□	2-⅞"□	2-1"□	3-1"□	3-1⅛	4-1⅛"□
	Bar G		none	none	none	none	none	½"□	½"□	½ □	½ □
	Distance Z							12"	10"	8"	6"
QUANTITIES	With min wings 18 Roadway	Cu Yds Conc	2078	2458	2873	3345	3845	4420	5041	5719	6410
		Rein Steel	730	835	900	1485	1706	2117	2970	3505	4150
	With max wings 18 Roadway	Conc	2215	2705	3208	3698	4222	4867	5522	6192	6914
		Steel	764	896	983	1609	1842	2336	3277	3854	4550
	Add [illegible] for each wing per ft change in [illegible]	Conc	548	6.8	670	706	754	813	875	946	1008
		Steel	36	153	166	248	272	398	558	698	800
	Add [illegible] per ft change in Roadway	Conc	472	573	683	768	852	937	1022	1107	1192
		Steel	1742	1955	2170	3440	3900	4715	6530	7420	8520

Note Above quantities include handrail over wings which add 1 cu yd concrete and 102# steel to each abutment (See slab and girder details)

Figure 143 b

TABLE B

Table of dimensions and quantities for design shown on figure 142.

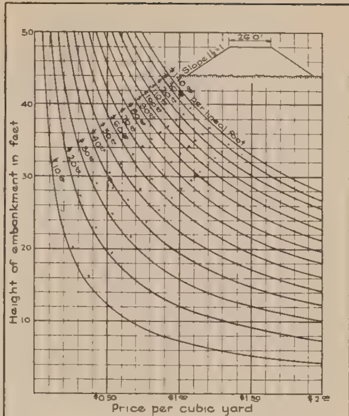


Figure 144

CURVES SHOWING COST PER LINEAL FOOT OF EMBANKMENT

Level sections - Heights 0' to 50'

Top width 26 0' 1½ to 1 slopes

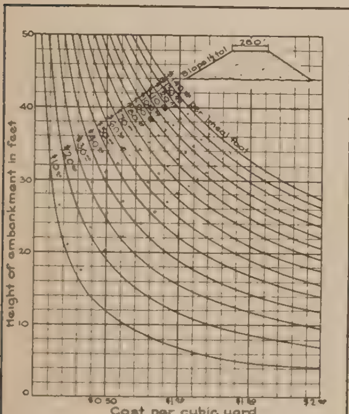


Figure 145

CURVES SHOWING COST PER LINEAL FOOT OF EMBANKMENT

Level section, 0 to 50

Top width 28.0' ~ 1½ to 1 slope

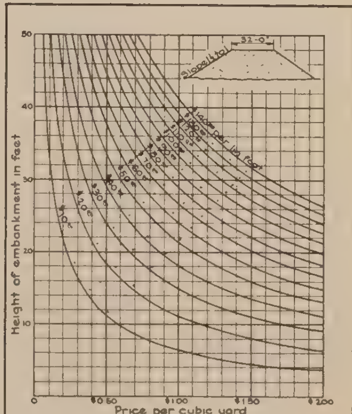


Figure 146

CURVES SHOWING COST PER LINEAL FOOT OF EMBANKMENT

Level sections, 0 to 50

Top width 32.0' ~ 1½ to 1 slopes

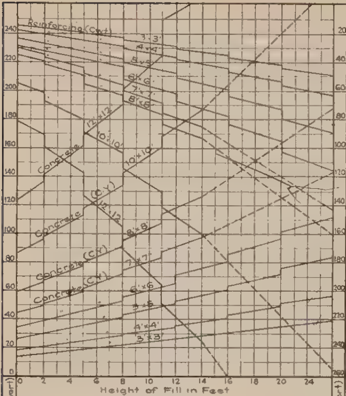


Figure 147a

QUANTITY CURVES FOR SQUARE CONCRETE BOX CULVERTS

24' Roadway and 1½ to 1 slopes on fills

Note At point where breaks occur in curves use the smaller amounts

Size		24'-0" Inside to Inside of Headwalls		One Lineal Foot of Barrell	
Height	Span	Concrete	Reinf	Concrete	Reinf
2	4	11 66	1170	.345	33 34
3	4	15 35	1462	384	37 36
3	6	21 40	1883	597	4514
3	8	28 20	2476	826	74 59
4	5	20.50	1876	532	4942
4	6	23 63	2038	644	55 23
5	8	34 60	2 906	927	72 38
5	12	54 84	3 912	1.560	128 40
6	8	39 10	3 298	969	75 20
8	12	72 60	4 910	1760	147 90

Figure 147b

TABLE OF QUANTITIES IN RECTANGULAR CONCRETE BOX CULVERTS

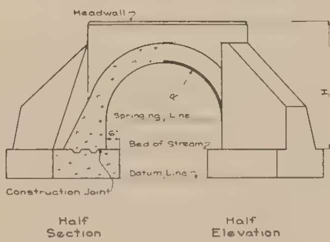


Figure 148

SKETCH SHOWING CONCRETE ARCH CULVERT

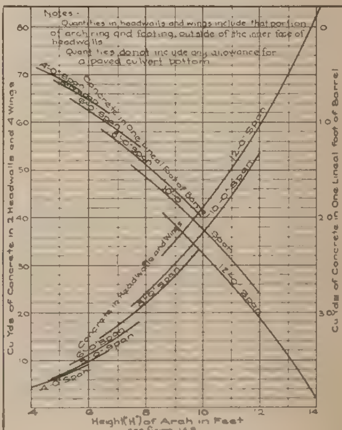


Figure 149

QUANTITY CURVES FOR CONCRETE ARCH CULVERTS

Note On low fills where handrails are required add for each lin ft of handrail 0.04 cu yds concrete and 3.2 lbs steel

of economics. The maintenance and renewal costs are practically nil for filled approach construction. This condition must, of course, be taken into consideration in any economic comparison.

Article 26.—Culvert Types

The various culvert types in common use throughout the country may be grouped as follows:

1. Reinforced concrete box culverts.
2. Concrete or masonry arch culverts.
3. Small circular concrete culverts.
4. Pipe culverts.
5. Other culvert types.

A typical standard design for reinforced concrete box culvert construction may be used in single span or in double or triple span.

Figure 147a gives the quantities of concrete and reinforcing metal needed for square culverts of this type for various spans, while Figure 147b is a table of quantities for rectangular box culverts.

A type of plain concrete *arch* culvert very similar to that adopted by the Pennsylvania State Highway Department is shown in Figure 148. Quantities for this type of culvert construction are given in Figure 149. The quantities given in Figure 149 provide for a footing depth varying from 1.0 foot for the smaller sizes, to 2.0 feet for the larger sizes. Where natural foundation material of the requisite bearing capacity and erosive resistance is not encountered at this depth, the above quantities must, therefore, be increased correspondingly.

A type of circular concrete culvert used for small openings is shown in Figure 150, this figure being reproduced from a design standard used by the State of Iowa.

Under the heading of pipe culverts may be listed the following types in most common use:

1. Concrete pipe culverts.
2. Corrugated metal pipe culverts.
3. Vitrified clay culverts.
4. Cast iron culverts.

A rather exhaustive series of experiments have been conducted at Ames, Iowa, under the direction of the Iowa State College Engineering Experiment Station, and other associated organizations, relative to the amount of load transmitted to culvert pipes under varying conditions as regards the placement of superimposed fill. It was found that two distinct conditions may exist in this regard, as follows:

A.—A "ditch" condition wherein the culvert is placed in a trench excavated through solid material and back filled with looser material.

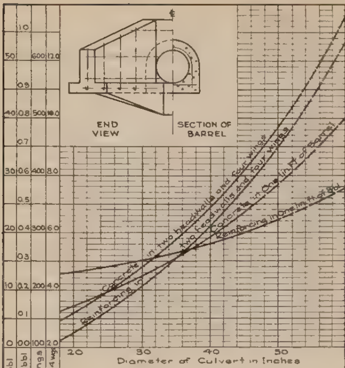


Figure 150

QUANTITY CURVES FOR CIRCULAR CONCRETE CULVERTS

Note
Quantities for headwalls and wings include that portion of barrel lying outside of the inner face of the headwalls

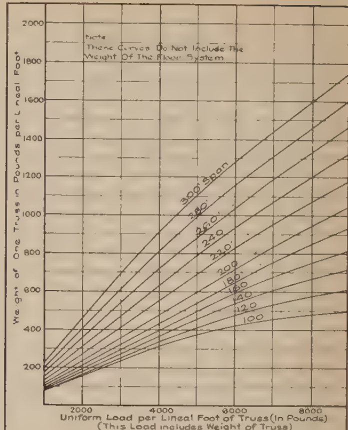


Figure 151

CURVES SHOWING WEIGHTS OF STEEL IN ONE TRUSS OF RIVETED THRU TRUSS BRIDGES FOR VARYING LOADS AND SPANS

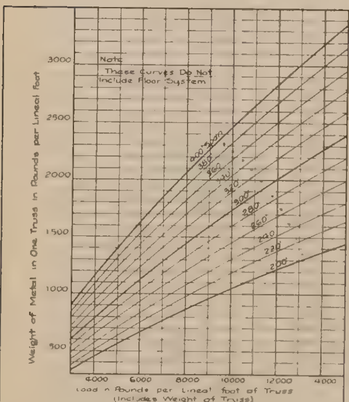


Figure 152

CURVES SHOWING WEIGHTS OF STEEL IN ONE TRUSS OF PIN CONNECTED PRATT TRUSS SPANS FOR VARYING LOADS AND SPANS

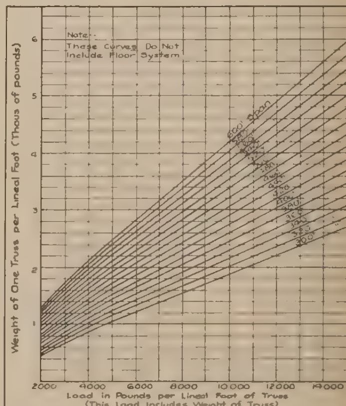


Figure 153

CURVES SHOWING WEIGHTS OF STEEL IN ONE TRUSS OF RIVETED SUB-PANELLED TRUSS SPANS FOR VARYING LOADS AND SPANS

B.—A "projection condition" wherein the entire fill, including both that portion lying directly above the culvert pipe and those portions lying adjacent thereto, is placed in one operation.

In the former case (ditch condition), the trench back fill over the culvert pipe being more loosely consolidated, has a tendency to deflect more under its own weight or that of any superimposed load. As this deflection progressively occurs, the sides of the ditch exert a restraining action, owing to the friction between the earth particles composing the sides of the trench against those composing the sides of the back fill. This frictional restraint operates to cause a certain portion of the trench load to be transmitted directly to the side of the trench, thus relieving the culvert of a portion of the total weight of the back fill.

In the projection condition, the situation is reversed. Here the material lying immediately over and above the culvert, being of less depth than the adjacent fill, deflects less. The adjacent fill, therefore, as it moves downward, transmits a certain portion of its load into the section lying immediately above the culvert pipe, so that the culvert pipe is called upon to carry not only the entire weight of all the fill placed about it, but also a portion of the adjacent fill transmitted thereto by friction.

The above discussion has an important bearing on the question of type selection for culvert pipe as will be observed in the paragraphs which follow.

Taking up first the concrete pipe culvert type, Tables A and B given below are of interest as indicating the appallingly high value of the unit stresses resulting from a theoretical stress calculation for pipe of the ordinary commercial sizes under various heights of embankment. This table is for the shell thickness as indicated and represents ordinary commercial practice for the lighter type of concrete pipe. Table C has been prepared along the same general lines, but is for the heaviest type of concrete pipe ordinarily procurable. It will be noted that the shell thicknesses in the second table are considerably greater and the unit stresses are correspondingly reduced. However, even in this case, the stresses for the larger culvert pipes are completely out of reason. As substantiating the high value of the theoretical stresses computed and tabulated in Tables B and C above mentioned, field observations covering a large number of pipe culvert installations disclose a rather general condition of crack formation for all of the larger sized pipe under heavy fills.

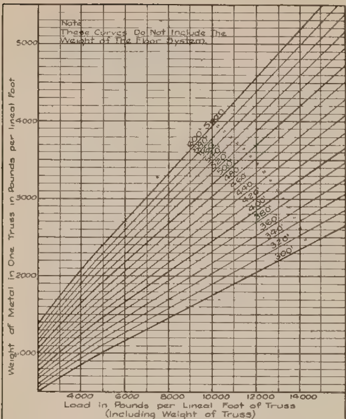


Figure 154
CURVES SHOWING
WEIGHTS OF STEEL IN ONE TRUSS
OF
PIN CONNECTED SUB-PANELLED TRUSSES
FOR VARYING LOADS
AND SPANS

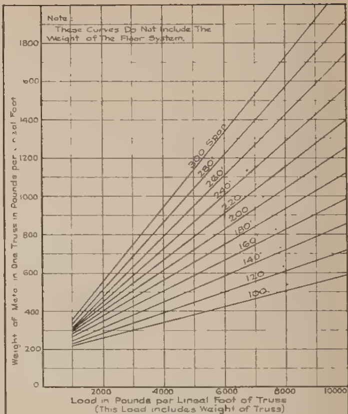


Figure 155
CURVES SHOWING
WEIGHTS OF STEEL IN ONE TRUSS
OF
STEEL DECK TRUSS BRIDGES
FOR VARYING LOADS
AND SPANS

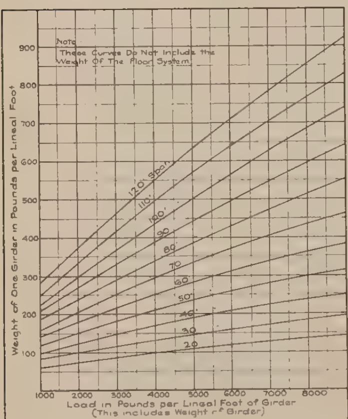


Figure 156
CURVES SHOWING
WEIGHTS OF PLATE GIRDERS
FOR VARYING LOADS
AND SPANS

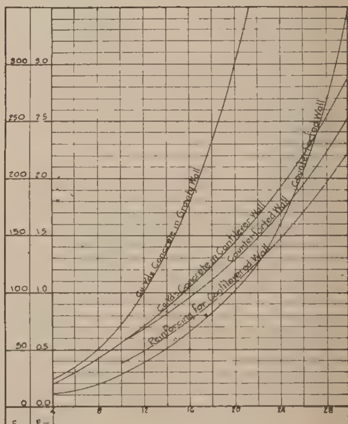


Figure 157
QUANTITY CURVES
FOR
ONE LINEAL FOOT OF RETAINING
WALL OF VARIOUS TYPES

Toe Pressure not to exceed 3 tons per square foot

TABLE A

BENDING MOMENT IN INCH-POUNDS PER LINEAL FOOT OF PIPE
PROJECTION CONDITION

Inside dia. in.	5 ft. fill	10 ft. fill	15 ft. fill	20 ft. fill
12	1,670	3,660	5,550	7,710
15	2,470	5,410	8,600	11,460
18	3,210	7,240	11,350	15,530
24	5,090	12,110	18,880	26,000
30	7,230	17,430	28,100	38,600
36	9,480	23,800	38,450	52,800
42	12,300	30,800	51,500	69,500
48	16,100	39,100	62,000	88,000

TABLE B

STRESSES IN POUNDS PER SQUARE INCH ON CONCRETE AND STEEL FOR STANDARD CULVERT PIPE

USING BENDING MOMENTS FROM TABLE A

Inside dia. in.	Shell thickness in.	Area of reinf. sq. in.	Concrete stresses 5 ft. fill	10 ft. fill	15 ft. fill	20 ft. fill	Steel stresses 5 ft. fill	10 ft. fill	15 ft. fill	20 ft. fill
12	2	.0	208	457	694	932				
15	2¼	.04	1390	3050	4860	6450	60,000	131,000	209,000	278,000
18	2½	.05	1370	3090	4840	6620	52,800	119,000	187,000	253,000
24	2¾	.08	1590	3780	5890	8120	51,800	123,000	192,000	264,000
30	3	.08	1950	4720	7580	10430	67,100	162,000	261,000	358,000
36	3¼	.11	2040	5120	8260	11320	61,300	154,000	249,000	342,000
42	3½	.13	2140	5370	8970	12100	59,700	147,000	250,000	338,000
48	4	.13	2260	5490	8700	12350	67,800	165,000	262,000	371,000

TABLE C

STRESSES IN POUNDS PER SQUARE INCH ON CONCRETE AND STEEL FOR CONCRETE PIPE OF EXTRA THICKNESS

AS SHOWN AND WITH INCREASED REINFORCEMENT, USING MOMENTS FROM TABLE A

Inside dia. in.	Shell thickness in.	Area of reinf. sq. in.	Concrete stresses 5 ft. fill	10 ft. fill	15 ft. fill	20 ft. fill	Steel stresses 5 ft. fill	10 ft. fill	15 ft. fill	20 ft. fill
15	3	.09	698	1530	2430	3240	20,400	44,700	71,000	95,000
18	3½	.13	615	1387	2160	2970	15,000	34,000	53,000	72,000
24	4	.26	620	1475	2300	3170	11,000	27,000	42,000	58,000
30	4½	.42	620	1497	2410	3320	9,000	22,000	35,000	48,000
36	5	.51	643	1615	2610	3580	9,000	22,000	35,000	48,000
42	5½	.56	690	1730	2890	3910	9,400	23,000	39,000	53,000
48	6	.61	760	1845	2930	4150	10,400	25,000	40,000	57,000

Table D is a compilation of stresses in standard corrugated metal culvert pipe for diameters varying from twelve to forty-eight inches and for fill heights ranging from five to twenty feet. It will be observed that with the exception of the smaller sizes and the lower fill heights, the stresses in every case run completely beyond the ultimate strength of

the metal. A study of this table would naturally lead to the belief that corrugated culvert pipe of the dimentions shown would immediately fail when placed under the load of a superimposed earth fill. The reason that complete failure does not take place is, of course, due to the fact that the culvert shell elongates along a horizontal diameter, taking an elliptical form with a shorter vertical axis. This operates to induce a settlement of the back filling over the culvert, thus transforming the "projection condition" into one greatly similar to the "ditch condition" hereinabove described. This action operates to greatly relieve the stresses in the metal.

TABLE D

STRESSES IN POUNDS PER SQUARE INCH ON STANDARD CORRUGATED IRON CULVERT PIPE

USING MOMENTS FROM TABLE A

Nominal diam. in.	Gauge	Stress under fill of 5 ft.	10 ft.	15 ft.	20 ft.
12	16	16,700	36,600	55,500	77,100
18	16	32,100	72,400	113,500	155,300
24	16	50,900	121,100	188,800	260,000
30	14	58,000	139,000	225,000	309,000
36	14	76,000	191,000	308,000	423,000
42	12	70,000	182,000	294,000	398,000
48	12	93,000	224,000	355,000	504,000

Table E indicates the stresses induced in standard vitrified clay sewer pipe, for diameters ranging from twelve to forty-two inches and for fill heights ranging from five to twenty feet. It will be noted that in this case also the theoretically computed stress under the "projection condition" is considerably in excess (especially for high fills) of the safe unit working stress in the material.

TABLE E

STRESSES IN POUNDS PER SQUARE INCH ON STANDARD VITRIFIED CLAY SEWER PIPE

Inside diam. in.	Thickness in.	Stress under fill of 5 ft.	10 ft.	15 ft.	20 ft.
12	1.0	840	1830	2780	3860
18	1.5	711	1610	2520	3450
24	2.0	635	1520	2360	3250
30	2.5	580	1290	2250	3190
36	2.75	625	1570	2530	3480
42	3.00	684	1715	2860	4900

In view of the foregoing, it may readily be concluded that none of the pipe culvert types above described can be regarded as truly permanent construction. It may be possible in the fill to simulate the "ditch condition" above described by a system of retrenching. It is questionable, however, if such a procedure is practicable as a field method. It would

be difficult to insure the requisite depth of trenching except under the most rigid inspection and in view of the fact that pipe culverts must, in many instances, be installed without individual inspection, it would hardly be safe to depend upon this being done in such a manner as to affect any great amount of stress relief.

It may be regarded as a safe prediction that pipe culverts of the above type will need renewal and replacement in many instances, owing to failure under service, so that as a class they cannot be regarded as in any sense the equivalent of the circular cast-in-place concrete culvert, indicated in Figure 150.

In Table F below is given a summary of the stresses in standard cast iron pipe culverts for diameters ranging from twelve to forty-eight inches and for fill heights from five to twenty feet. The shell thickness given in the second column is for the standard minimum weight of commercial cast iron pipe. The thicknesses given in the last three columns are those necessary to reduce the unit stress to a value of 12,000 pounds per square inch. The low value of stresses given in Table F are in direct contrast to those for the other types of culvert pipe hereinabove described. It will be noted that only for the extreme size and for the heavier fills is there any need for a shell thickness greater than the commercial minimum. Even for this size it will be noted that the thickness required (as given in the last three columns) is well within the limits of the commercial thicknesses readily procurable in cast iron pipe.

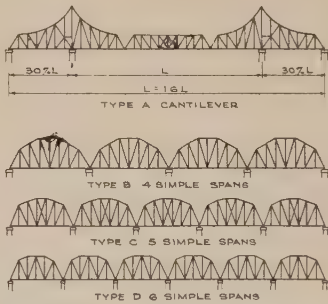
TABLE F

STRESSES IN POUNDS PER SQUARE INCH FOR STANDARD CAST IRON PIPE, CLASS A, AND THICKNESSES REQUIRED IN ORDER TO KEEP UNIT STRESSES DOWN TO 12,000 POUNDS PER SQUARE INCH

USING MOMENTS FROM TABLE A

Inside Dia. In.	Shell Thick-ness In.	Stress under fills of 5 ft.	10 ft.	15 ft.	20 ft.	Thickness required under 10 ft. fill	15 ft. fill	20 ft. fill
12	0.54	2,900	6,300	9,500	13,000			.57
18	0.64	3,900	8,800	13,800	19,000		69	.80
24	0.76	4,400	10,500	16,300	22,400		88	1.04
30	0.88	4,700	11,300	18,100	25,000		1.08	1.27
36	0.99	4,800	12,200	19,700	27,000	1.00	1.27	1.49
42	1.10	5,100	12,700	21,300	29,000	1.13	1.46	1.71
48	1.26	5,100	12,300	19,500	28,000	1.28	1.61	1.92

Summarizing the above discussion, therefore, it may be concluded that the permanent solution for pipe culvert installation offers a choice between a cast iron pipe fitted with adequate headwalls, where necessary, or else a circular cast-in-place concrete culvert of the type indicated in Figure 150. Other pipe culvert types, while doubtless serving a useful purpose



Span L	Total Length	Total weight of steel in Cantilever as a percentage of total weight of		
		TYPE B	TYPE C	TYPE D
400	640	180%	208%	225%
500	800	180%	211%	232%
600	960	180%	214%	239%
700	1120	180%	217%	246%
800	1280	180%	220%	253%
900	1440	180%	223%	260%
1000	1600	180%	226%	267%

Figure 158

and undoubtedly much cheaper in first cost, must, in view of the above discussion, be regarded largely in the light of temporary or at best, semi-permanent construction.

Article 27.—Retaining Walls

Quantity curves for one lineal foot of retaining wall for various heights from four feet to thirty feet are plotted in Figure 157 for three types of design, viz.: the gravity or mass type, the cantilever type and the counter-forted type. These quantities have been computed from designs wherein the unit toe pressure has been limited to about three tons per square foot maximum and the earth pressure taken as equivalent to that for a fluid weighing thirty (30) pounds per cubic foot.

Article 28.—General Observations Regarding Use of Curves and Cost Data

The cost and quantity data given hereinabove do not by any means cover every type of construction likely to be encountered in practice, but only the most commonly used types. It may be possible, however, to use the curves given hereinbefore as a basis for the determination of data in many instances falling entirely outside the scope of the curves themselves. For example, it may be that the quantities are desired for a steel span designed for a load specification entirely different from those for which

these weight curves have been plotted. In this case, if the weight of *any one given span length* can be determined and plotted as a point upon the weight curves given herein, a parallel curve may be sketched in from which the weights of other span lengths designed for the loading in question may be determined with a reasonable degree of accuracy.

The above procedure may also be followed when it is desired to determine the weights of spans having roadway widths outside the limits of the given curves. In this latter case, extrapolation methods may also be employed with reasonable accuracy.

If weights of spans carrying sidewalks are desired, approximate results may be obtained from the given curves by considering the width of the sidewalk equal in loading value to an additional roadway width of 65% of the sidewalk width for spans wherein the sidewalks are placed inside the trusses and equal to 30% where the sidewalks are cantilevered. Results given by the above method are, of course, only approximate.

In order to furnish data wherein steel truss and girder spans for other loading conditions may be roughly estimated, the curves shown in Figures 151 to 156 have been included. These curves have been plotted for different span lengths and loading conditions and give, for any given span and total load per lineal foot of truss or girder, the approximate weight of metal in the trusses or girders, exclusive of floor system and laterals. Figure 151 is for riveted Pratt truss spans from 100 feet to 300 feet in length. Figure 152 is for pin connected Pratt trusses in spans from 200 feet to 400 feet in length. Figures 153 and 154 are for subpanelled trusses riveted and pin connected, respectively. Figure 155 is for deck truss spans and Figure 156 for plate girders. The data given by these curves are roughly approximate only and have been compiled from various published data for railway loadings by a process of reduction to meet highway design practice, as regards details and metal thickness requirements and should not be used where the necessary quantity data may be obtained from the other curves.

Article 29.—Swing Bridges and Cantilever Spans

The determination of weights for movable bridge construction is without the province of this chapter and in general may be treated as a special problem.

For rough estimates, however, it may be assumed that the metal in a swing draw span of 100 feet total length is about the same as that for a simple span of the same length. From this value, the weights of swing spans decrease gradually and rather uniformly to a value, for spans having a total length of 500 feet, equal to about 80% of that for a simple span of the same total length.

The above figures are for rim bearing swing spans. Center bearing swings will run from 3% to 10% less.

The weights of cantilever bridges vary greatly with the type of design and arrangement of spans. Figure 158 illustrates one of the most frequently used span arrangements and gives the approximate weights in terms of percentages of various arrangements of simple spans designed for the same loading. These data are very incomplete, but may be used in the absence of better information as a basis for rough preliminary estimates.

Article 30.—Unit Prices

The data which form the subject matter of the articles in this chapter which have gone before have had to do largely with the matter of quantities of material for the different construction types. In order to convert these items into terms of first cost, it becomes necessary to apply unit prices. In general the relative quantities of material in any two span types measure the relative unit cost. This is not true, however, when the construction types under comparison are such as to render different unit prices applicable. For example, mass construction in concrete may involve a yardage considerably in excess of reinforced construction and still take a unit cost that will render the total first cost less than that of the lighter reinforced type.

In order to apply unit costs, which may be in approximately correct ratio for different construction types, the following table has been prepared from the average of a large number of cost data obtained from the writer's cost records. In this table, the unit costs for various types of construction in timber, steel and concrete are given as a percentage of the cost of the least expensive corresponding type. This table is, of course, a rough average of many results and must be used with discretion. It will, however, yield a very fair idea of the relative unit costs to be expected.

COST RATIOS FOR VARIOUS TYPES OF CONSTRUCTION
TIMBER CONSTRUCTION

Type of construction	Percentage unit costs based on 100% for trestles		
	Material f. o. b. site	Labor	Total
Trestle spans	60	40	100
A-frame spans	65	45	110
Unhoused truss spans	75	75	150
Housed truss spans	70	65	135

STEEL CONSTRUCTION

Type of construction	Percentage unit costs based on 100% for I-beam spans: Material f. o. b. site	Fabrication	Erection and painting	Total
I-beam spans	70	12	18	100
Plate girder spans	70	17	23	110
Truss spans	70	22	28	120

CONCRETE CONSTRUCTION

Type of construction	Percentage unit costs based on 100% for mass concrete: Material f. o. b. site	Mixing and placing	Form material	Form labor	Finish	Total
Mass concrete	60	20	15	5	0	100
Heavy reinforced walls and slabs	65	20	25	23	2	135
Standard beam and girder construction	65	25	30	40	5	165
Light open spandrel arch construction and other ornamental work	65	25	35	60	5	190

CHAPTER V

RENEWAL AND MAINTENANCE COSTS

ARTICLE 1.—INTRODUCTORY

Having presented the data in Chapter IV needed for the evaluation of the term C representing the first cost for various construction types, it now becomes necessary to consider the evaluation of the terms M and R of the economic equations given in Chapter III.

Throughout all of the discussion which comprises the subject matter of this chapter, it has been assumed that maintenance work is being carried on regularly, periodically and systematically under the direction of an organized bridge maintenance department and that bridges are regularly inspected and maintained through a system of periodic repairs, renewals and reinforcements. If this is not done, there is, of course, no method or basis upon which to compute maintenance costs, nor any way in which to arrive at an estimate of such costs.

ARTICLE 2.—CLASSIFICATION OF MAINTENANCE CHARGES AND AVERAGE ANNUAL COSTS

In order to arrive at any definite basis for the determination of annual bridge maintenance costs in reference to construction types, it is first necessary to segregate these cost items under several distinct classifications. The following segregation has been adopted by the writer's office and is presented as being the most logical arrangement possible, in view of the data at hand.

CLASS 1—Maintenance Costs Common to all Bridges.— Under this classification may be grouped those items of maintenance work, the necessity for which is due to conditions which are independent of the type of construction used. This group may be further divided into two sub-classifications. The first sub-class comprises those items of maintenance work which are necessary from the standpoint of traffic service. Typical of this class of work items are the following:

1. Protection of approach embankment shoulders. In explanation of this item, it may be said that with modern traffic concentration,

some little difficulty is experienced in maintaining a full roadway width and unimpaired riding qualities at the junction between approach fill and structure proper. It has been found necessary in many instances to protect the shoulders by means of handplaced riprap or concrete block masonry in order to maintain full shoulder widths and to prevent subsidence under the impact of traffic. Where roadways are surfaced, this procedure also eliminates in a large degree, the tendency for the surfacing to settle and crack at the junction between fill and bridge.

2. Slope protection at ends of span. This item of work is similar to Item No. 1, except that the protection is carried the full height of the embankment slope and heeled into the natural ground at the toe of the slope to prevent erosion. Item No. 1 is generally necessary in every case. Item No. 2 may or may not be necessary, depending upon conditions.

3. Catch basins and drains at ends of span. In explanation of this item of work it may be observed that the concentration of drainage water on the bridge floor and its discharge at the end of the span, operates in many cases to cause the formation of gullies or washes along the face of the adjacent embankment slopes. In extreme cases, it is necessary to construct catch basins and drains to take care of this situation. Weep holes or drain scuppers through the bridge floor proper take care of a portion of this water it is true, nevertheless, a great deal of slope erosion results from this cause unless properly maintained as described herein.

The above items of maintenance are typical of a large number of minor work-items which are necessary in the case of nearly every type of construction and constitute, in a sense, a fixed maintenance charge which is quite constant. The shoulder protection will cost not to exceed $20.00 per shoulder in the first instance, or $80.00 per bridge. The slope protection will vary in cost between $150.00 and $500.00 per slope, or between $300.00 and $1,000.00 per bridge. Catch basins and drains will add from $25.00 to $50.00 to the cost of the shoulder protection.

The above figures represent the total cost in the first instance. In order to convert such figures into terms of annual maintenance cost, it is necessary to distribute the same over their probable economic service period. Since this work is comparatively permanent, it is probably conservative to estimate that the annual total expenditure for this purpose will average about $60.00 per year per bridge. In addition to the above distributed first cost, there will be a small maintenance charge to cover minor repairs to the slope pavements and drains.

The above repairs in connection with such other items as are in general common to all construction types, will increase the total so that a figure of $100.00 per bridge per annum is probably not far from correct.

The second sub-classification under this main class includes such items as are rendered necessary by hydraulic conditions. Among these may be mentioned the following:

1. The placement of stream bed pavements, curtain walls, cut-off walls, etc., to protect foundations from erosion.
2. Channel excavation incident to the clearing and straightening of stream channels.
3. The placement of foundation riprap.
4. Other items of like nature.

It is impossible to give any figure which will cover the above work, as the same varies between wide limits, depending upon individual conditions. For properly designed construction, such expense items as large channel changes, revetments, wing walls, dikes, etc., are presumably taken care of in the first cost of construction. Furthermore, stream bed pavements are presumably placed as part of the original construction where needed and all foundations are carried to such depths as to eliminate the necessity for curtain walls, underpinning, riprap, etc. It is apparent that the greater the degree of care exercised in the original design and construction, the less will be the cost of such maintenance items as are enumerated in this classification. On the other hand, it is unquestionably true that work of this character in any kind of construction, necessitates a certain amount of expenditure each year. This expenditure constitutes a fixed maintenance charge and probably should be taken into consideration in any economic analysis.

There are other fixed maintenance charges which must be taken into consideration in this connection, among which may be mentioned the following:

1. Engineering supervision for maintenance work.
2. Administration and overhead, rental on idle maintenance equipment, etc.
3. Cost of annual inspection of bridge structures for maintenance purposes.
4. Other items of like nature.

In view of the foregoing and of the complications involved, it is rather futile to attempt any close determination of fixed maintenance costs of this type. Maintenance cost records kept in the writer's office over a period of years indicate a total value covering all of the fixed maintenance charges enumerated hereinabove under the general designa-

tion of Class 1 Maintenance, to vary between 0.1% and 0.5% per annum. For the purpose of economic analysis, it is probably close enough, therefore, to assume a fixed maintenance charge equal to the average of the above figures or 0.3% per annum for all spans regardless of type.

CLASS 2—Maintenance Costs Common to Timber Trestle Spans.—Under this classification may be included such items of maintenance work as the following:

1. Repairs and renewals to deck, as follows:
    a. Deck member repairs and renewals.
    b. Repairs and renewals to wearing surfaces.
    c. Repairs and renewals to hand railing.
    d. Repairs and renewals to wheel guards.
    e. Repainting hand rails, wheel guards, etc.
    f. Other miscellaneous minor repairs.
2. Renewals and replacements of stringers, caps, posts, bracing, etc.
3. Piling renewals.
4. Creosote oil brush treatment and painting.
5. Cutting off old timber piling and blocking over same.
6. Building concrete pedestals to support trestle posts used to replace decayed piling.
7. Placing new bolts.
8. Miscellaneous minor repairs, such as tightening bolts, general repairs to caps, sills, posts, etc.
9. Placement of new bridging, repairs and renewals to bulkheads, etc., etc.

The total annual cost of this class of work increases with the age of the structure. It is, of course, difficult to give an exact figure which may be used to represent average annual maintenance costs. Figure 1 has been plotted from maintenance cost records in the writer's office and represents actual annual maintenance costs for the type of trestle construction illustrated and described in Chapter IV. The records unfortunately do not extend beyond a period of fifteen years service life, the curves beyond this point being sketched in as a projection or prolongation of the record curve.

Few trestle structures are maintained in service for a period in excess of twenty years and for this reason it is not probable that a great deal of maintenance cost data for trestle spans in excess of this age will ever be collected. As the structures continue in service beyond the time period indicated in Figure 1, a greater degree of constant deterioration from decay will be apparent as the various fungi producing rot in any structural timber show a tendency to spread out inducing contamination of

adjacent sticks. New timber used as replacements in old construction will, therefore, have a shorter life than otherwise. Not only decay but a general condition of decrepitude resulting from vibration stresses and other causes operate to increase the maintenance on trestle construction until it is probable that a condition would be reached wherein 10% of the structural members would require renewal each year. The unit costs for replacements are always greatly in excess of like unit costs for new construction. The work must be done under traffic and generally the amount involved at any one particular time is so small as to render the overhead cost extremely high. The above 10% renewal, therefore, would undoubtedly represent a value in excess of 10% of the total first cost and the values indicated in Figure 6 may be safely assumed as representing the upper limit for annual maintenance charges when trestle structures are carried on and on indefinitely through a system of maintenance that in reality partakes of the nature of fractional replacements.

It will be noted that Figure 1 contains two curves, one being plotted through the center of gravity of those results which indicate a minimum maintenance cost and the other through the center of gravity of those results which indicate a condition of maximum maintenance cost. These two curves represent the probable variation in maintenance charges for standard trestle construction of the type described in Chapter IV. The difference in cost is partly explained by climatic conditions, partly by local conditions and partly by the type of trestle construction itself. For an extremely humid climate (such as the coastal regions) and for pile construction, especially Class B construction, the maximum maintenance rates would apply. For standard Class A frame construction on concrete pedestals and under arid or dry climatic conditions the minimum maintenance costs might be expected.

The curves given in Figure 1 have been plotted from records which include Class 1 maintenance as well as Class 2.

CLASS 3—Maintenance Costs Common to Timber Truss Spans.—Under this classification may be placed such items as the following:

1. Tightening bolts.
2. Adjustment of camber.
3. Renewals and reinforcements for splices.
4. Placement of new bolts.
5. Repacking of chords.
6. Reinforcement of joints.
7. General renewals to truss members, floor system and decks.
8. Other miscellaneous minor repairs.

For unhoused truss construction the average maintenance costs as determined from the writer's cost records for spans of the type described in Chapter IV are indicated in Figure 2. The maximum maintenance curve represents Class B construction, construction in coastal regions (or other localities where humidity conditions are high) or construction of timber which due either to the quality of the timber itself or to local conditions is found to season check badly under service or to warp or twist out of bearing. The minimum maintenance costs will occur for Class A type construction and for structures where climatic and other local conditions are favorable to long service life.

It will be observed that the maintenance curve contains a rather pronounced "hump" at the age of one to two years; that is due to the fact that truss construction in timber nearly invariably shows a tendency to become sagged or to lose its camber after the first year's service due to shrinkage and a general compression of all the joints. The counters become loose and the vertical tension rods lose their relative adjustment. For the foregoing reasons it is generally necessary to go over each truss within a year or so after construction and "tune it up" as it were. After this work is done once or twice the truss seems to hold camber much better and no additional work of this nature need be done for a long time.

For housed timber trusses the items enumerated above are necessary but at intervals which are considerably less frequent. The fact that the truss members are *housed in* protects them from decay to a marked extent. The housing also has the effect of shading the timber and maintaining it under comparatively uniform temperature conditions. All of these operate to prolong the life of the individual structural members and thus greatly lessen the maintenance costs. As a partly offsetting charge housed truss construction will require the following additional maintenance items from time to time:

1. Repairs and renewals to shingle roofs.
2. Repairs to housing.
3. Interior and exterior painting.
4. Staining of shingle roof.
5. Other miscellaneous items resulting from the use of housed construction.

Figure 3 is a maintenance cost curve for housed truss construction taken from the maintenance cost records above mentioned. These represent maintenance costs for construction such as described in Chapter IV. The "hump" at the one and two-year periods due to the necessity for "picking up the camber" as above discussed is apparent in this curve as well as in the case shown in Figure 2.

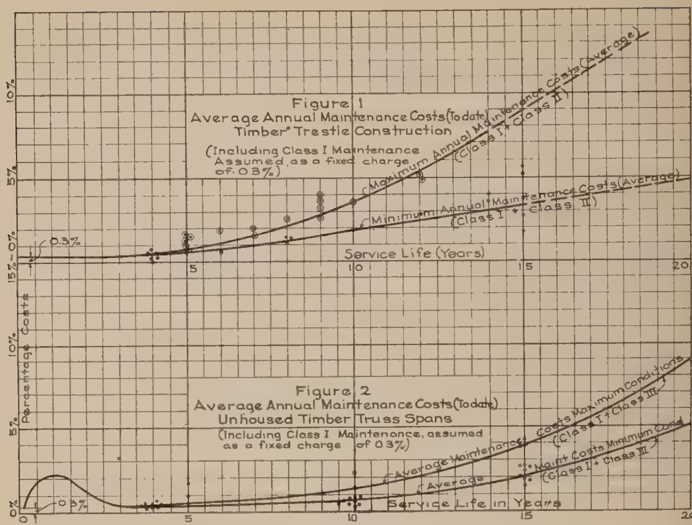


Figure 1
Average Annual Maintenance Costs (To date)
Timber Trestle Construction
(Including Class I Maintenance Assumed as a fixed charge of 0.3%)
Maximum Annual Maintenance Costs (Average) (Class I + Class II)
Minimum Annual Maintenance Costs (Average) (Class I + Class II)
Service Life (Years)
0.3%
Figure 2
Average Annual Maintenance Costs (To date)
Unhoused Timber Truss Spans
(Including Class I Maintenance, assumed as a fixed charge of 0.3%)
Average Maintenance Costs Maximum Conditions (Class I + Class III)
Average Maint Costs Minimum Cond (Class I + Class III)
Service Life in Years
0.3%
Percentage Costs

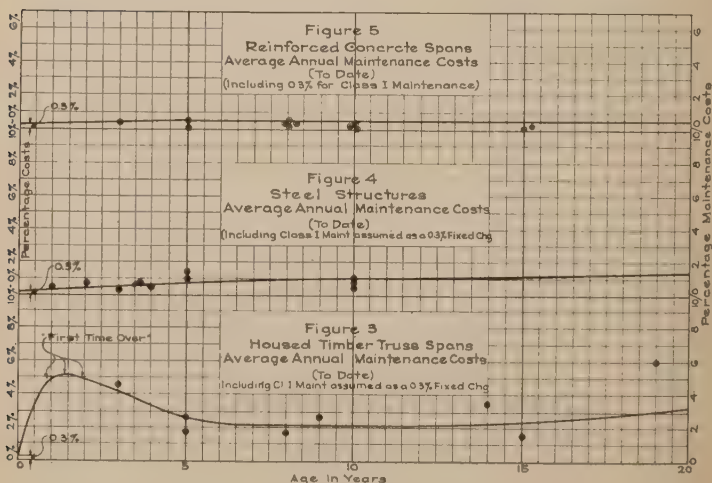


Figure 5
Reinforced Concrete Spans
Average Annual Maintenance Costs
(To Date)
(Including 0.3% for Class I Maintenance)
0.3%
Figure 4
Steel Structures
Average Annual Maintenance Costs
(To Date)
(Including Class I Maint assumed as a 0.3% Fixed Chg)
0.3%
Figure 3
Housed Timber Truss Spans
Average Annual Maintenance Costs
(To Date)
Including Cl I Maint assumed as a 0.3% Fixed Chg
"First Time Over"
0.3%
Percentage Costs
Percentage Maintenance Costs
Age in Years

CLASS 4—Maintenance Costs Common to Steel Spans.—Figure 4 is a curve of annual maintenance costs for steel structures of the general type described in Chapter IV. The principal item of work coming under this classification is the repainting of the structural steel. For localities exposed to salt laden atmosphere or to injurious gases this repainting must be done at two to three year intervals. For interior localities once in four to five years is generally sufficient. The curve represents average conditions, for extreme conditions the percentage costs should be increased 30% to 50%. In addition to the item of painting there are sundry minor items such, for example, as maintenance of roller nests, expansion joint renewals, maintenance on floors and surfacing and at rather long intervals replacement of loose rivets.

CLASS 5—Maintenance Costs Common to Concrete Construction.—The statement has frequently been made that one of the advantages for concrete construction was its freedom from maintenance expense. This is not exactly true. However, maintenance costs for concrete construction are less than that for any other construction type. With modern highway traffic, considerable damage is being done to concrete wheelguards and handrails by heavy motor trucks. The rails are being fractured due to collision with loaded trucks in many instances, in other cases the damage is an abrasion or gouging of the surface, resulting from some sharp object comprising an overhanging load. This difficulty has necessitated an expenditure for cleaning and patching concrete railings and for repairs and renewals to rails, rail caps and other members, such as precast handrail posts, lamp posts, etc., composing the handrail or balustrade.

In addition to the foregoing, there arises the necessity in many instances for the cleaning and patching of the concrete surfaces (especially where the construction has been such as to leave certain of the reinforcing metal with a rather light cover) and for certain minor items of maintenance, such as the opening of weep holes in concrete floors, the cleaning out of concrete culvert barrels, the removal of drift and debris from around piers, etc., etc. In general these items are very small individually; altogether, however, they constitute a fixed maintenance charge that probably should receive some consideration.

Figure 5 indicates the average annual maintenance expense for concrete construction over a fifteen year period taken from the above mentioned maintenance cost records (including Class 1 maintenance). To all intents and purposes a fixed maintenance charge of 0.6% may be considered as applicable for the first 20 years of service life.

Article 3.—Estimated Future Maintenance Costs

The curves of Figures 1 to 5 are plotted from actual cost records but unfortunately do not extend over a period of time greater than 15 to 20 years. In order to arrive at a better basis for the logical calculation of maintenance costs Figure 6 has been prepared, the same being an extension of the curves of Figures 1 to 5 for a forty year future service period. The method by which the values for maintenance costs for trestle structures have been extended has already been dwelt upon. The curves for the unhoused truss spans have been extended in a similar manner; in this case as in the case of trestle construction it has been assumed that the yearly maintenance costs will continue to increase as the structure continues to grow old, but that a 15% annual maintenance cost would represent a maximum, being a condition wherein any structure could be carried on indefinitely year after year, being in effect *renewed piece by piece.* This, of course, is not a logical nor an economical procedure since the cost of piece meal renewals is always abnormally high, moreover, structures under a condition of this kind are quite apt to become obsolete as regards traffic service requirements. These curves (Figure 6) must, therefore, be regarded as representing a hypothetical condition rather than a real one and are of value only as forming a basis for the calculation of economic service life for various types as described in the next article.

It seems reasonable to assume that maintenance costs would eventually run as high as 15% per annum on old unhoused construction. The cost of maintenance as these spans grow in age is very high. Decay in new timber is always much more rapid when placed as a renewal in old construction due to a spreading of the fungi spores from adjacent, partially decayed timber. The structure becomes loosened and vibratory under continual service; decks wear out rapidly and an increasing amount of work is necessary at joints and splices owing to the tendency for the timber to warp, season check and twist out of bearing. For construction of this type the element of danger enters in to a greater extent than in trestle construction, since a failure of a truss member is likely to cause complete structural collapse with its attendant likelihood of loss of life and property damage. For this reason it becomes increasingly necessary as the spans grow older that the individual members, the joints, the details and the splices be very rigidly maintained.

For housed construction conditions are vitally different. In this case decay is checked to such an extent as to render it safe to assume a service life for each individual stick used as a replacement of twice and perhaps three times the corresponding value for replacements in unhoused con-

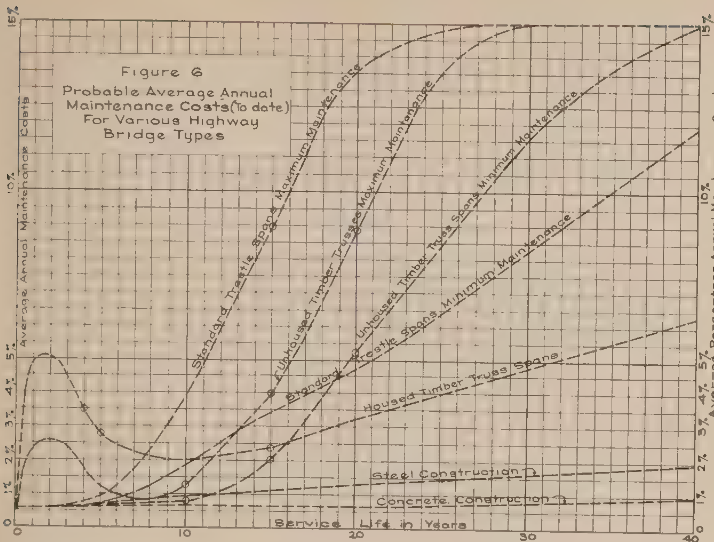


Figure 6
Probable Average Annual Maintenance Costs (To date) For Various Highway Bridge Types
Average Annual Maintenance Costs
Average Percentage Annual Maintenance Costs
Standard Trestle Spans Maximum Maintenance
Unhoused Timber Trusses Maximum Maintenance
Unhoused Timber Truss Spans Minimum Maintenance
Standard Trestle Spans Minimum Maintenance
Housed Timber Truss Spans
Steel Construction
Concrete Construction
Service Life in Years

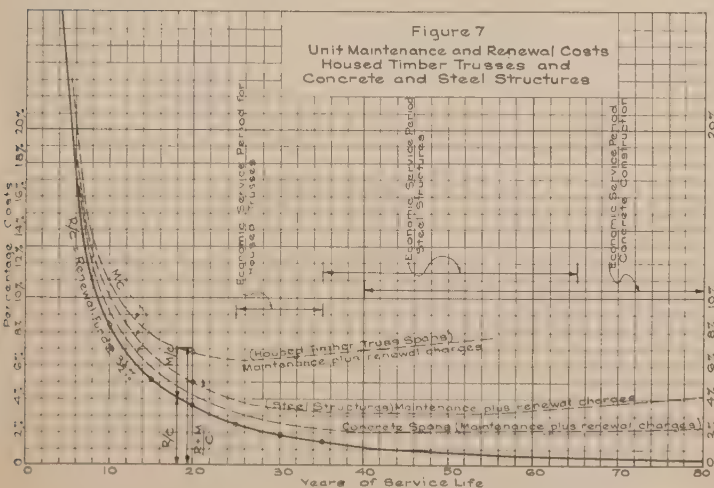


Figure 7
Unit Maintenance and Renewal Costs Housed Timber Trusses and Concrete and Steel Structures
Percentage Costs
Economic Service Period for Housed Trusses
Economic Service Period Steel Structures
Economic Service Period Concrete Construction
(Housed Timber Truss Spans) Maintenance plus renewal charges
(Steel Structures) Maintenance plus renewal charges
Concrete Spans (Maintenance plus renewal charges)
Renewal
Years of Service Life

struction. The shingles and housing need frequent renewal it is true and this brings up the maintenance costs to some extent, but 6% to 7% seems to represent about what may be considered the extreme upper limit.

The maintenance costs on steel spans may increase slightly as the span grows old owing to the increasing number of loose rivets, channelled pins, etc., and to the need for sand blasting or scraping in connection with repainting operations. Probably, however, this cost need never exceed 2%.

The maintenance costs on concrete spans will probably increase to about 1% owing to a general tendency to disintegrate at expansion joints and to a wear on roadway surfaces, etc.

In general, therefore, it may be considered that the average maintenance costs given in Figure 6 represent fairly well the conditions which may be expected for the various types over a period of years.

Article 4.—Renewal Costs

In order to arrive at a basis for the computation of the annual renewal cost for any particular construction type, it is first necessary to determine the economic service life of the structure in question. The term *economic service life,* as hereinafter used, designates the time period for which the combined cost of maintenance and renewal charges is a minimum. This value may be readily determined by superimposing the annual maintenance cost curve upon a curved base representing the unit renewal cost. The procedure in detail is as follows:

Lay off, as abscissae, the age of the structure in years and for each year plot, as ordinate, the amount which, when deposited annually for this given number of years, at the assumed rate of compound interest, would accumulate the sum of one dollar. Clearly, from the discussion which has been given in Chapter III, this curve represents the unit renewal cost $\frac{R}{C}$ for any structure whose service life is represented by the abscissa to the ordinate in question.

Upon this curve, as a base, superimpose the unit maintenance cost curves given in Figure 6. The total ordinate to this last curve clearly represents the value $\frac{R+M}{C}$ and the economic service period for the structure is clearly the point of minimum ordinate for the combined curve.

Figures 7 and 8 have been plotted for the bridge types given in Figure 6 and are doubtless self-explanatory. The renewal fund has been calculated upon the basis of 3½% interest (either 4 or 4¼ may be used as these rates are often used, however, the resulting curve would not differ greatly from the one shown in Figures 7 and 8). If desired new curves may be prepared for different interest rates.

From the curves of Figures 7 and 8 the following data are at once obtained:

Type of construction	Range of economic service life	Unit renewal and maintenance costs $R + M \div C$
Timber trestles (most unfavorable conditions)	10-12 yrs.	12.2%
Timber trestles (most favorable conditions)	18-20 yrs.	8.0%
Unhoused trusses (most unfavorable conditions)	12-15 yrs.	9.0%
Unhoused trusses (most favorable conditions)	15-17 yrs.	7.3%
Housed truss spans	25-35 yrs.	6.2%
Steel spans	35-65 yrs.	3.2%
Concrete construction	40-80 yrs.	2.1%

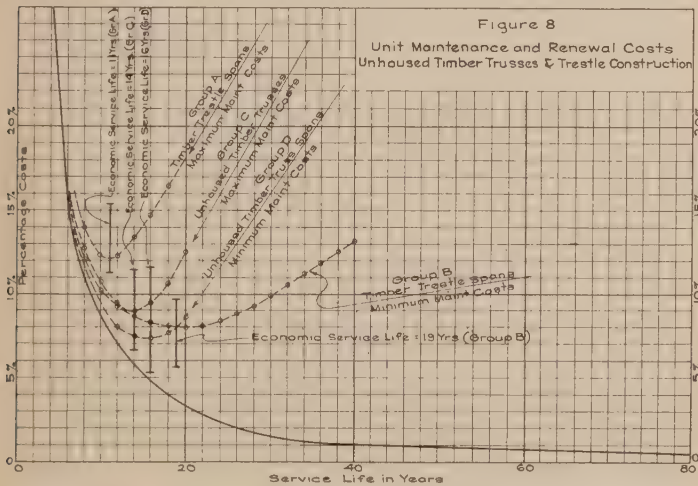


Figure 8
Unit Maintenance and Renewal Costs
Unhoused Timber Trusses & Trestle Construction

The various types of construction enumerated in the foregoing table will be briefly considered in the order given.

Timber Trestle Construction.—It will be noted that the economic service period for this type of construction varies between the limits of ten and twenty years, this difference being a direct reflection of the variance in maintenance cost. It will also be noted that the percentage cost for maintenance plus renewals varies between the limits of 8% and 12.2% The above variation in service value is due in part to local and climatic conditions and also to a considerable extent to the class of construction adopted. In localities where the humidity is high, as in the coastal regions, service periods will be relatively short. In arid

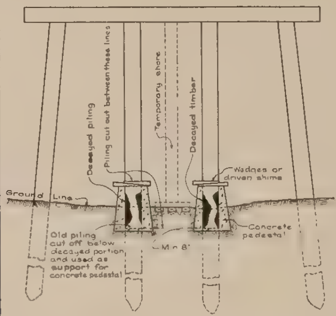


Figure 9
Concrete pedestals used as a method of maintenance for piling decayed at ground line

climates, the service period will be considerably longer, unless climatic conditions are such as to cause a rapid deterioration from undue season checking, warping or twisting of the timber in service. As between types, the pile trestle construction will have a shorter life than the frame trestle construction on concrete pedestals, under ordinary conditions, due to the fact that the maintenance of piling involves either a redriving under traffic, which is comparatively high priced work, or a replacement of the decayed portion of the pile, by means of blocking or the use of concrete pedestals, as indicated in Figure 9. Either of these conditions introduces a maintenance cost comparatively large in magnitude. In view of the foregoing, therefore, the service life of pile trestle construction is quite apt to be less than for frame trestle construction.

Where frame construction is used, and where conditions are favorable to erosion, considerable maintenance cost is entailed in the protection of the concrete pedestals. Figure 10 illustrates a condition often encountered in instances of this kind and a condition involving the necessity for riprap or underpinning at a comparatively high unit cost. In cases of this kind, therefore, the maintenance cost for pile construction will be less than for frame construction and the economic service life will be proportionately greater.

The type of details used in the trestle design will also have an impor-

Figure 10. Undermined pedestal on frame trestle construction.
(A condition which renders the pile trestle more economical for eroding bottoms.)

tant bearing upon the maintenance cost and consequently upon the economic service life.

The following design features have a tendency to lessen maintenance costs:

1. Bolted felloe guard connections.
2. Rigid anchorage of stringers and decking.
3. Heavy laminated decking with adequate horizontal nailing or better yet a tongue and groove detail thus converting the deck into a rigid unit.
4. Adequate provision for drainage of decking.
5. Adequate provision for protection of the timber at all water pockets or portions of the structure which are apt otherwise to retain moisture.
6. Flared turnouts at the ends of the handrailing which protect the roadway shoulders at the ends of the bridge and also protect the end of the railing from damage.
7. Bolted connections throughout which facilitate replacements with minimum labor costs and permit members to be removed and replaced without the danger of splitting as in the case of nails and boat spikes.

8. Bulkhead details which protect the ends of the floor system from contact with earth, or damp timber.
9. Bulkhead details keyed to the end bent in such manner as to prevent damage due to traffic impact.
10. Provision for adequate ventilation of all timber where possible.
11. Creosoting or other preservative treatment for all contact surfaces of timber against metal, earth or other timber.
12. Other details of like nature.

A marked increase in service life is also the result of a comparatively massive stringer and deck system, such structures being less likely to deteriorate from impact and vibration stresses. The use of heavy construction with a large factor of safety, also operates to increase the time period between visits by the maintenance crew. This last is true because of the fact that in construction of this kind a slight occurrence of decay or any other slight deterioration will not cut into the limiting safe strength to as great a degree as in the case of lighter construction. Renewals in this class of work, therefore, can be made at less frequent intervals and maintenance costs considerably reduced, thereby.

Throughout the foregoing, it has been assumed that the construction under discussion is such as has been described in Chapter IV. If a lighter or more insubstantial type of construction is used, the economic service periods given in the above table will be correspondingly reduced. It is probable that for the light and insubstantial construction types formerly adopted in many localities for highway trestle construction, the service life should be assumed as having not over 60% of the above tabular values. The unit maintenance costs should, of course, be correspondingly increased.

It has also been assumed throughout the foregoing discussion, that the above structures are subject, at all times to a rigid system of periodic inspection and maintenance. If this is not the case, the economic service life will be reduced to a value not greater than 70% of the value given above.

Uncovered Timber Truss Spans.—From the table above it will be noted that the range in service life for this type of construction is from twelve to seventeen years, depending upon the same condition as described hereinabove for trestle construction, as regards climatic conditions. In reference to the effect of the details upon maintenance costs (and therefore upon economic service life) it may be said that deck trusses being more rigidly braced and, to a certain extent, protected from the weather, will have a longer life than through trusses. The following details in design will also operate to reduce maintenance costs:

1. The use of cast angle blocks. In this connection it should be observed that the first cost is greatly increased by the use of cast angle blocks as indicated from the curves given in Chapter IV, so that the question of economic utility is one which must be determined in each individual case. It is probable that, except for structures carrying Class A loading and for structures in particularly damp localities, the interest on the additional first cost of cast angle blocks constitutes an offsetting charge greater than the annual saving in maintenance cost.
2. The use of heavy metal splices for all tension chords.
3. The use of details which will permit camber adjustment with the minimum outlay of expense.
4. The adoption of details which will permit access to all joints and details for inspection and will permit of the maximum number of renewal or maintenance operations without the necessity for swinging scaffolding.
5. The use of long shear tables at splices and joints. These should, in general, be designed with a factor of safety at least 50% greater than that used for the main members.
6. Other structural details as enumerated hereinabove under trestle construction.

Covered Timber Truss Bridges.—From the above table, it will be observed that the economic service life for this type of construction is about double that for equivalent construction where unhoused. The reason for the reduced maintenance cost in the case of housed construction lies in the protection afforded the timber from moisture and from adverse conditions of temperature. The length of time over which housed timber will remain free from decay or deterioration is rather surprising to those who have not made it a point to investigate. For example, the truss shown in Figure 54 of Chapter IV was over 45 years old when finally taken down and although the housing was in a very dilapidated condition, due to inadequate maintenance, the truss members were all entirely sound and free from decay of any kind. Throughout some of the eastern states, such for example, as Pennsylvania, there are many covered timber highway spans still standing, the original construction of which antedates the Civil War. Dr. Waddell in his treatise on Bridge Engineering, mentions a cantilever bridge built in Wandipore, Tibet, having a span of 112 ft. and being constructed entirely of timber, fastened together with wooden pegs, no metal of any kind being used in the structure. This structure, according to Dr. Waddell, rendered service for a period of 150 years. Another housed timber structure, built at Waterford, New York, in 1804.

rendered service for a period of 105 years, or until the year 1909, at which time it was destroyed by fire.

Steel Construction.—With organized and systematic maintenance, steel construction should be capable of rendering service over a comparatively long period. From the table hereinabove given it is noted that the average economic service period appears to vary between the limits of 35 years to 65 years. The general deterioration in construction of this type is due to a loss of section through corrosion and a general loosening up of the structure due to the development of loose rivets, channelled or worn pins, etc. As regards corrosion losses it may be said that, in general, the actual amount of deterioration from this cause has probably been overestimated. During the year 1923 the writer had occasion to take down some old steel tubular piers which had been used to support a highway truss superstructure over Pudding River between Marion and Clackamas Counties, Oregon. There were two sets of tubes, one set being of wrought iron and the other of ordinary steel. The wrought iron tubes had been in place for 31 years, while the steel tubes were only eleven years old. Neither set of tubes had apparently been painted since the date of erection nor had any attempt at protection from corrosion of any kind been made. The tubes passed through the natural ground line above the elevation of low water but below high water elevation, so that at the ground line the tubes, during any season, were alternately wet and dry. It was considered, therefore, that, all in all, conditions were very favorable to a very rapid corrosion which impression was borne out by the external appearance of the metal which appeared badly tubercled and covered with a heavy rough oxide scale. To determine the actual rate of corrosion loss, samples 12″ x 12″ were cut from the worst spots, brushed and pickled in acid. These plates were weighed as received, and also after the acid treatment and the latter weight compared with the original theoretical weight computed from the measured thicknesses for portions of the tube showing no evidence of corrosion. These weights were also checked against the weights of plates of equal size and of the nominal thickness of the original metal. Eleven samples were tested in this manner and the probable corrosion loss computed in terms of inches per year. The following is a summary of results:

Material	Location	Average corrosion loss in inches per year
Wrought iron	Above ground	0.00018
Wrought iron	At ground line	0.00046
Wrought iron	Below ground line	0.00045
Steel	Above ground	0.00042
Steel	At ground line	0.00094
Steel	Below ground line	0.00075

As would naturally be expected the corrosion loss was very much more rapid in the case of steel than for the wrought iron. Also the maximum loss occurred at or near the ground line, this is also what would ordinarily be expected since conditions at the ground line are distinctly more favorable to corrosion than those at any other portion of the tube. The feature worthy of note in this connection, however, is the relatively low value of the annual corrosion loss. The worst condition above noted results in a total loss of metal for a one hundred year period of less than 0.1 inch in thickness. If the above can be considered typical, the maximum loss in sectional area for metal structures designed under a $\frac{5}{16}$ inch minimum specification would amount, during the maximum service period given in the table hereinabove (65 years), to less than 20% even if no painting of any description were to be done. Of course, it cannot be assumed that corrosion will never progress at a greater rate than that above noted; as a matter of fact the writer has observed instances wherein $\frac{3}{16}$ inch lattice bars and webs of channels 0.2 of an inch in thickness have been completely rusted through during a period of 18 to 20 years. However, in view of the foregoing (especially when the utter lack of any maintenance whatsoever in the above recorded instance is borne in mind) it may be safely concluded that the deterioration of metal from corrosion will not be at a rate rapid enough to seriously limit its service life, provided that maintenance painting is done systematically, under suitable skilled supervision and at sufficiently frequent intervals. The character of climatic conditions has an important bearing upon the service life of steel construction. Salt atmosphere, or atmosphere laden with locomotive or other injurious gases, stimulates corrosive action and, therefore, increases the cost of maintenance painting. The type of construction also bears an important relationship to maintenance expense. In general, I-beam spans and deck plate girder and truss construction may be carried with the smallest expense of any steel superstructure construction; first, because the metal is to a certain extent protected from the weather and second, because in general such construction is more rigidly braced than through construction and, therefore, less apt to be affected by the vibratory effects of traffic. Rigid, riveted construction will have a longer service life than light pin connected structures for the reason hereinabove set forth. Structures with concrete floors will outlast those designed for timber floors and structures designed under a $\frac{5}{16}$ inch minimum specification will also outlast those having $\frac{1}{4}$ inch metal for plates and minor parts.

Concrete Construction.—The range of service life given for concrete construction in the above table is from 40 to 80 years and the

average annual cost for maintenance plus renewals is given as 2.1%. From a study of the curves of Figures 7 and 8 it is observed that the combined cost of maintenance and renewals does not vary much over this range in service life so that for purposes of economic comparison it does not make a great deal of difference which service life (between the above limits) is assumed.

As a matter of fact the service period for mass construction, designed and executed under adequate engineering supervision, might be regarded as almost indefinite. In this case it may also be true that the maintenance charge above assumed is rather high, however, data are not at hand to warrant any greater degree of refinement.

For the lighter types of reinforced concrete construction (post and girder viaducts, for example) it may be that after a service life of forty years (or perhaps fifty years) a general deterioration of the concrete at the edges and corners or perhaps a rusting of the reinforcing metal underneath the concrete will operate to increase the annual maintenance costs above 2.1% per annum. Again, it may be said that data at hand are insufficient to warrant a conclusion.

For concrete exposed to the action of sea water or in soils which are strongly alkaline the above maintenance cost figures do not, of course, apply, these being in the nature of special cases.

It should be remembered not only in connection with concrete structures but also in reference to any of the construction discussed throughout this chapter that the figures given herein are based upon structures designed for modern highway traffic needs, with careful attention given to detail and with sections adequately proportioned for all stresses likely to be imposed. Construction of this type has been illustrated throughout Chapter IV. If the construction in question is of a lighter type, insubstantially designed and with less attention paid to detail throughout, maintenance costs will be proportionately increased.

It should also be borne in mind that all assumptions made hereinabove concerning future service periods have been based upon the futher assumption that the structures are adequately proportioned for future traffic needs, so that they will not become obsolete before they are worn out. In other words, it is assumed that roadway widths, traffic capacities, locations and other like features, as well as strength requirements have been selected with a view to future traffic needs, so that the service period will be determined by the deterioration of the construction from a purely physical standpoint.

Figure 11

Constants for Solution of Equation

Total Annual Cost $C\left[r+M/C+R/C+I/C-P\right]+\left[O+O'\right]$

(R-computed on basis of $3\frac{1}{2}\%$)

Type	M/C %	R/C %	I/C %	Interest @ 5% $[r+M/c+R/c+I/c-p]$				Interest @ 6%			
				P=2%	P=1%	P=½%	P=0	P=2%	P=1%	P=½%	P=0
Timber Trestles Group I (Conditions favorable to minimum service life)	3 7	8 5	0 2	15 4	16 4	16 9	17 4	16 4	17 4	17 9	18 4
			0 3	15 5	16 5	17 0	17.5	16 5	17 5	18.0	18.5
			0 5	15 7	16 7	17 2	17 7	16 7	17 7	18.2	18.7
Timber Trestles Group II (Conditions favorable to max service life)	4 0	4 0	0 2	11 2	12 2	12 7	13 2	12.2	13.2	13 7	14 2
			0 3	11 3	12 3	12 8	13 3	12 3	13 3	13.8	14 3
			0 5	11 5	12.5	13.0	13.5	12 5	13 5	14 0	14 5
Unhoused Timber Trusses Group I (Conditions favorable to min service life)	2 0	7 0	0 2	12,2	13 2	13 7	14 2	13 2	14 2	14 7	15 2
			0 3	12 3	13.3	13 8	14 3	13 3	14 3	14 8	15 3
			0 5	12 5	13 5	14 0	14 5	13.5	14 5	15 0	15 5
Unhoused Timber Trusses Group II (Conditions favorable to max service life)	2.0	5 3	0.2	10 5	11.5	12.0	12 5	11 5	12 5	13 0	13 5
			0.3	10.6	11.6	12.1	12 6	11 6	12 6	[illegible]	13 6
			0 5	10.8	11.8	12.3	12 8	11 8	12 8	13 3	13 8
Housed Timber Truss Spans.	3.0	3.2	0 2	9 4	10.4	10.9	11 4	10.4	11 4	11 9	12 4
			0 3	9 5	10.5	11 0	11 5	10 5	11 5	12 0	12 5
			0 5	9 7	10 7	11 2	11 7	10.7	11 7	12 2	12 7
Steel Construction	2 2	1.0	—	6 2	7 2	7 7	8.2	7 2	8 2	8 7	9 2
Concrete Construction	1 4	0.7	—	5.1	6.1	6 6	7 1	6.1	7 1	7 6	8.1

Article 5.—Percentage Coefficients for Solution of Economic Equations

The above discussion completes the data necessary for the determination of all the unknown terms used in the economic analysis equations given in Chapter III. These, with the exception of the operating costs, have all been converted into *percentage coefficients* which may be applied directly to the first cost C of any proposed construction type when this first cost has been determined from the curves and data of Chapter IV.

The table given in Figure 11 is a summary of these coefficients for two interest rates (5% and 6%), for three different rental and insurance values and for renewal charges computed at 3½%. This table may obviously be extended to cover other insurance, rental, interest or renewal rates if local conditions are such as to make these applicable.

Article 6.—Fundamentals of Type Analysis

Before concluding this chapter it should perhaps be pointed out that the entire method of type comparison hereinbefore outlined is based upon two factors, viz.: yearly maintenance costs and traffic operation costs. Once these are determined the other factor coefficients fall into place as it were.

To illustrate, let it be required to determine the relative economy of a reinforced concrete deck versus a 6-inch laminated timber deck for a 200-foot steel span under the following conditions as regards design, unit first costs, and volume of traffic.

Length of structure.............................200 feet

Width of roadway................................ 20 feet

Cost of 6-inch laminated timber deck in place......$50.00 per M

Cost of reinforced concrete deck in place
(including reinforcing metal)...............$33.00 per cu. yd.
Average traffic ....500 vehicles per day
Thickness of concrete deck (average)..................7 inches

From the above figures the following additional data may be developed:

Approx. total cost of 6-inch laminated timber deck
(including wheel guards)........................$1,280.00
Approx. total cost of reinforced concrete deck
(including curbs) $3,420.00

The first step in the analysis is the determination of the probable annual maintenance costs for the above types. These data are obtained from maintenance records where available and plotted in curve form. Figure 12 is a series of curves plotted through results obtained from the writer's maintenance cost records. The upper curve represents the average yearly maintenance costs on timber decks under the most adverse conditions; the lower curve represents average maintenance costs for timber decks under the most favorable conditions.

These curves are for timber decks wherein the service period is limited and determined by the deterioration of the timber due to decay rather than by traffic wear. Laminated decks, in general, resist traffic wear much better than plank decks owing to the fact that the decking pieces are laid with vertical grain. This type of decking under a traffic of less than about 500 to 600 vehicles per day (except for heavy trucking) will wear rather evenly and may be maintained at the costs indicated in Figure 12 until progressive deterioration from decay increases the maintenance cost above the point of economy. For plank decks under extremely light traffic this condition may also hold but, in general, for traffic above 200 vehicles per day the roadway surface becomes too rough for traffic use long before decay has seriously affected the decking pieces. The curves of Figure 12, therefore, would not apply to plank decks except for very light traffic.

With the above understanding the curves of Figure 12 may be used for the purposes of the problem at hand.

These yearly maintenance cost curves are next superimposed upon an annual renewal cost curve as a base. The results are given in Figure 13. The upper and lower curves of Figure 13 represent what may be regarded as the extreme range in conditions for laminated timber decks in the locality and for the traffic conditions covered by these data (for other conditions curves varying somewhat from these will doubtless be obtained). Now, it is apparent that for all intermediate conditions, the

combined cost of maintenance and renewal will be somewhere between these two extreme values as shown diagramatically by the dotted line in Figure 13. A study of the actual conditions in any individual instance will enable the engineer to make an interpolation reasonably close to the truth.

Let it be assumed that, for the conditions at hand, the most probable assumption as to yearly maintenance plus renewal costs is 12½%.

Maintenance and renewal costs for concrete construction are shown by Figure 11 to average 2.1%. This figure is for concrete construction in general. For concrete bridge floors the maintenance costs run somewhat higher so that a value of 3% for renewals plus maintenance may be assumed for the purpose of this problem. (If actual maintenance costs are available for concrete floors as a class, curves similar to Figures 12 and 13 should be plotted for the same.)

With the above data and assuming an insurance rate of 0.2% for the laminated decking the total annual costs to the state may now be calculated as follows (neglecting any consideration of rental values):

$$\text{Annual cost to state} = C\left[r + \frac{M+R}{C} + \frac{I}{C}\right]$$

For the two types of deck this equation reduces to the following:

Annual cost of laminated deck = \$1,280 [5% + 12.5% + .2%] = (\$1,280.00) (17.7%) = \$226.50.

Annual cost of concrete deck = \$3,420.00 [5% + 3%] = \$273.60.

From the standpoint of the state it is thus seen that the most economical construction is the timber decking. However, as discussed in Chapter III, the total annual cost of any construction must include not only those costs accruing to the state but also the costs accruing to *traffic*.

Using the figures given for traffic costs in Chapter III, the cost of operation may be calculated as follows:

For the laminated timber deck

$$O = (500)\ (365)\left(\frac{200}{5280}\right)(\$.102) = \$705.00$$

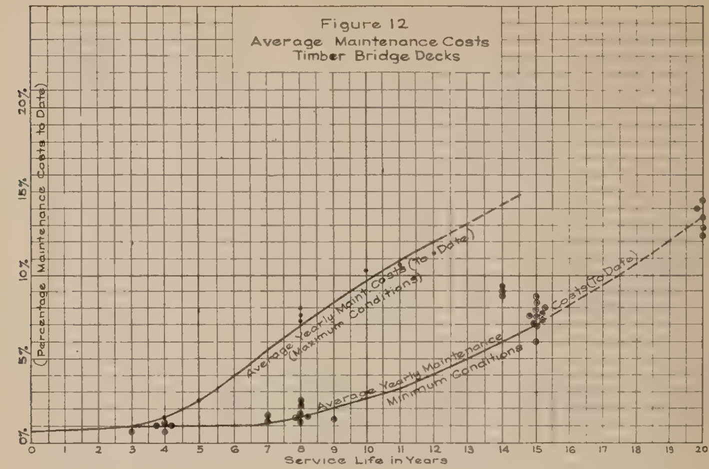
For the concrete deck

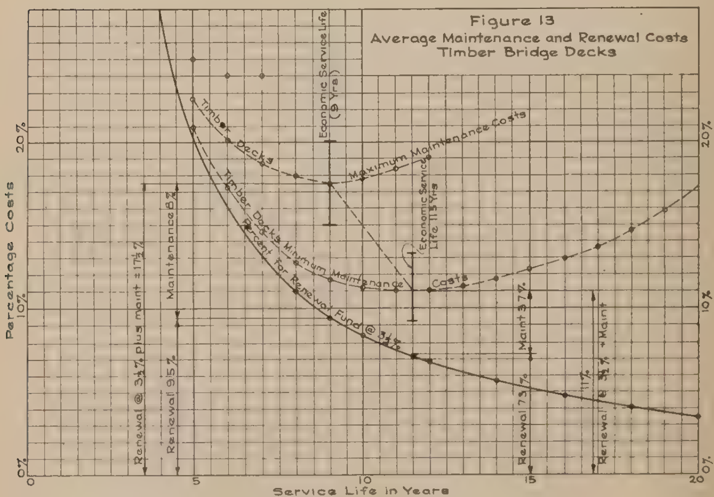
$$O = (500)\ (365)\left(\frac{200}{5280}\right)(\$.08) = \$554.00$$

From the above figures the following combined costs are derived:

For the laminated timber deck

Total annual cost = \$226.50 + \$705.00 = \$931.50.





For the reinforced concrete deck

Total annual cost = \$273.60 + \$554.00 = \$827.60.

If the above assumptions are correct the timber decking will prove

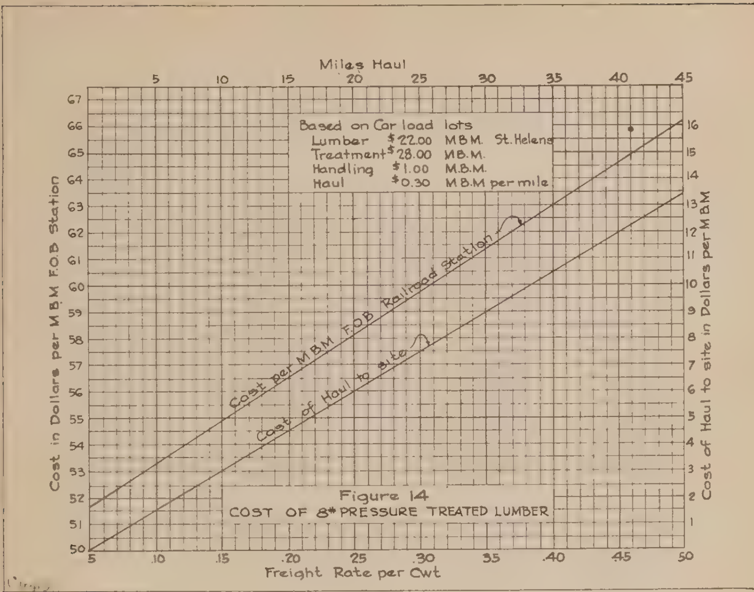


Figure 14
COST OF 8# PRESSURE TREATED LUMBER

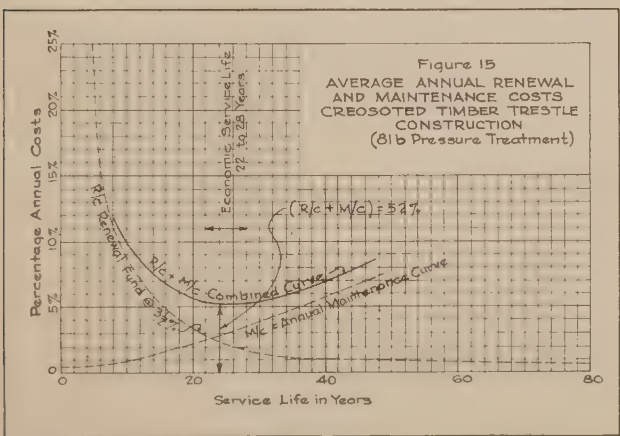


Figure 15
AVERAGE ANNUAL RENEWAL AND MAINTENANCE COSTS CREOSOTED TIMBER TRESTLE CONSTRUCTION
(8 lb Pressure Treatment)

economical until traffic has reached a density value D given by the equation:

$$\left[\frac{(365)(200)(.102 - .08)}{5280}\right] D = 273.60 - 226.50$$

or $D = 154$ vehicles per day.

The above analysis has been used simply to illustrate the proper method of attack and may or may not represent true conditions. It has been assumed that the choice between decking types lies between a 6-inch laminated timber deck and a reinforced concrete deck. This has been done in order to avoid a lengthy discussion. In actual practice, it is quite possible that other decking types would be found to deserve consideration. In any event, however, the method to be followed is the same as outlined above.

It is anticipated, therefore, that with the data given in the foregoing chapters, no difficulty should be experienced in making a correct economic analysis of competing construction types for any highway bridge problem ordinarily encountered in practice.

Article 7.—Economics of Creosote or Other Preservative Treatment for Timber

In the discussion which forms the subject matter of the foregoing paragraphs, no mention has been made of timber construction in which the timber has been treated by creosote or other preservative process.

Treated timber has been used in bridge construction by the railroads for many years and is fast gaining in popularity for highway bridge construction in localities where the price of untreated timber is sufficiently high to render the use of treatment truly economical.

There are several methods used for timber treatment and a discussion of each of these, even briefly, would doubtless be out of place at this point.

The process in most common use is that of treating the timber by means of creosote oil, which process covers in general three distinct methods of application, viz.:

(a) Pressure treatment.
(b) Open tank treatment.
(c) Brush coat treatment.

Taking up first the pressure treatment and without going into detail it may be stated that the process consists in first thoroughly seasoning the lumber either by air seasoning, steam seasoning or oil seasoning. Steam seasoning, which process is used for Southern Yellow pine, con-

sists in the placement of the timber in a steam cylinder at about 20 or 25 pounds pressure per square inch for not more than 8 hours for sawed timber and not more than 20 hours for piles. After the steaming is completed, a vacuum of about 22 inches is drawn and maintained until the wood is as dry and moisture-free as practicable. The oil seasoning process is used for Douglas Fir and consists in boiling the timber in oil *under a vacuum* until the moisture remaining in the wood is reduced to an amount such as will not prevent the injection and proper distribution of the creosote.

After the timber has been seasoned by one of the processes above described, the preservative is injected under pressure. The following extract from the standard bridge specifications of the A. A. S. H. O. will indicate the general trend of practice in this regard:

> The following pressure processes shall be used for the treatment of Southern Yellow Pine:
>
> (a) **Full-Cell Process (Oil Treatment).**—Timber shall be subjected to a vacuum of sufficient intensity and duration to insure that the wood is as dry and free from air as practicable and to permit a retention of the specified number of pounds of preservative per cubic foot of wood.
>
> The preservative shall be introduced between 165°F. and 200°F., and the cylinder filled without breaking the vacuum. The pressure shall then be raised to and maintained at a minimum of 100 lbs. per square inch or until the quantity of preservative required to insure the final retention stipulated is injected into the wood, or until the purchaser's representative is satisfied that the largest volumetric injection that is practicable has been obtained. The temperature of the preservative during the pressure period shall be not less than 150°F. nor more than 200°F., and shall average at least 180°F. After the pressure is completed the cylinder shall be emptied speedily of preservative, and a vacuum of not less than 22 inches promptly created and maintained until the wood can be removed from the cylinder free of dripping preservative.
>
> (b) **Empty-Cell Process with Initial Air. (Oil Treatment).**—Timber shall be subjected to air pressure of sufficient intensity and duration to provide under a vacuum the ejection of surplus preservative, and to insure a retention and proper distribution of the stipulated number of pounds of preservative per cubic foot of wood. The preservative shall be introduced between 165°F. and 200°F., the cylinder pressure being maintained constant until the cylinder is filled with preservative. The pressure shall then be raised to and maintained at a minimum of 150 lb. per square inch or until there is obtained the largest practicable volumetric injection that can be reduced to the

stipulated retention by a quick, high vacuum, or until the purchaser's representative is satisfied that the largest volumetric injection that is practicable has been obtained. The temperature of the preservative during the pressure period shall be not less than 150°F., nor more than 200°F., and shall average at least 180°F. After pressure is completed, the cylinder shall be emptied speedily of preservative, and a vacuum promptly created and maintained until the wood can be removed from the cylinder free of dripping preservative.

(c) **Empty-Cell Process without Initial Air. (Oil Treatment).**—The preservative between 165°F. and 200°F., shall be introduced to the timber until the cylinder is filled. Pressure shall then be raised to and maintained at a minimum of 100 lb. per square inch or until there is obtained the largest practicable volumetric injection that can be reduced to the stipulated retention by a quick high vacuum, or until the purchaser's representative is satisfied that the largest volumetric injection that is practicable has been obtained. The temperature of the preservative during the pressure period shall be not less than 150°F., nor more than 200°F., and shall average at least 180°F. After pressure is completed, the cylinder shall be emptied speedily of preservative and a vacuum of not less than 22 inches promptly created and maintained for not less than 30 minutes until the quantity of preservative injected is reduced to the required retention and the wood can be removed from the cylinder free of dripping preservative.

The following pressure processes shall be used for the oil treatment of Douglas Fir:

Heating with Oil.—Air-seasoned or kiln dried Douglas Fir will not be required to be boiled under a vacuum, but it may be desirable to hold the material in a creosote bath maintained at a temperature of 180° to 190°F. for a length of time which, combined with the pressure period, is in the judgment of the operator necessary to secure the specified absorption.

(a) **Full-Cell Process.**—Following the heating period in the case of air-seasoned material, and the seasoning under vacuum period in the case of material artificially seasoned, the cylinder shall be filled with creosote and pressure applied as required to a maximum limit of 175 pounds per square inch and maintained, taking into consideration the quantity of creosote absorbed during the bath, until the specified absorption of creosote has been obtained.

Temperature of the creosote during the pressure period shall be as high as possible, with a minimum limit of 160°F. and a maximum limit of 200°F.

After pressure is completed, the cylinder shall be emptied of creosote and a vacuum of at least 20 inches promptly created and maintained for a sufficient period of time to free the material of dripping creosote.

(b) **Empty-Cell Process with Initial Air.**—Following the heating period, in the case of air-seasoned material, and the seasoning under vacuum period in the case of material artificially seasoned, the material shall be subjected to air pressure of an intensity and duration which, in the judgment of the operator, is sufficient to accomplish the final retention of creosote specified.

The preservative shall then be introduced, the air pressure being maintained constant, until the cylinder is completely filled.

Creosote shall then be pressed from the measuring tanks into the wood in a quantity sufficient, in the opinion of the operator, to leave the required retention at the completion of the process herein described. Maximum pressure shall in no case exceed 200 pounds per square inch. The temperature of the creosote during the pressure period shall be as high as possible, within a minimum limit of 160°F. and a maximum limit of 200°F.

After pressure is completed, the cylinder shall be quickly emptied of creosote and a vacuum of at least 20 inches promptly created and maintained for such period of time as may be required to remove dripping creosote from the material.

The amount of preservative to be used shall be shown on the plans or specified and this amount shall be retained in the timber unless the oil has been injected to refusal. Unless otherwise specified, the amount of preservative retained shall be as follows:

(a) **Creosote or Creosote-Coal-Tar:**

1. For piles and timber in general bridge construction:
   Full-cell process, not less than 12 lb. of oil per cubic foot of timber, or
   Empty-cell process, not less than 8 lb. of oil per cubic foot of timber.

2. For piles or timber in salt water subject to attack by marine borers:

   Full-cell process.
   Southern Yellow Pine, not less than 20 lb. of oil per cubic foot of timber.
   Douglas Fir, not less than 15 lb. of oil per cubic foot of timber.

The cost of pressure treatment for Douglas Fir on the Pacific Coast may be indicated roughly by the following quotation recently received:

"We are pleased to indicate the following prices f. o. b. cars, our plant:

	Timbers per M.B.M.	Piles per cu. ft.
8 pound treatment	$26.00	$0.40
10 pound treatment	30.00	.50
12 pound treatment	35.00	.60
14 pound treatment	40.00	.70

"If the same quantity of oil is specified, there is no difference in the cost of full and empty cell treatment; therefore the above prices hold for both.

> "To determine the cost of piles per running foot, it is necessary to determine the cubiture per lin. ft. Care must be used in figuring this to allow a leeway over the minimum diameters. For example, we allow a taper of 1 inch to 10 lin. ft. and figure the stick will carry a butt 1 inch larger than the minimum diameter specified. To determine the cubiture of a 30 foot pile on an 8 inch top, 14 inch butt specification, we figure a 15 inch butt, 12 inch top, or 1 cu. ft. per lin. ft."

The cost of treated timber is made up of the following component costs:

Cost of lumber F. O. B. plant.
Cost of treatment at plant.
Freight on treated lumber to R. R. station nearest bridge site.
Handling at R. R. station.
Hauling to bridge site (truck or team haul).

Figure 14 has been prepared to indicate the total cost of treated timber F. O. B. the bridge site on the basis of the following assumptions:

Treatment—8 pound—empty cell process.
Cost of treatment—$28.00 per M.B.M.
Cost of lumber F. O. B. plant—$22.00 per M.B.M.
Handling at R. R. station—$1.00 per M.B.M.
Truck or team haul to site, 30 cts. per M.B.M. for each mile hauled.

For example, the total cost for a 30-cent freight rate and a 10-mile haul is as follows:

	Per M.B.M.
Cost F. O. B. station	$59.75
Cost of handling at R. R. station	1.00
Cost of haul	3.00
Total	$63.75

For any other known prices, the curves may be prorated to give the desired results. For example, if the lumber F. O. B. plant were to cost $25.00 per M, and the cost of treatment were $30.00 per M, the total cost of treated timber F. O. B. the bridge site in the above case would be:

Cost F. O. B. station ($59.75 + $3.00 + $2.00) =	$64.75	per M
Cost of handling at R. R. station	1.00	
Cost of haul	3.00	
Total	$68.75	per M

Whether or not the extra expense involved in creosote treatment is warranted is entirely a question of economics. As has been repeatedly stated hereinabove, this question in turn hinges upon the relative annual maintenance costs for the treated *vs.* the untreated types. Unfortunate-

ly, the writer does not have tabulated maintenance costs for the treated type of timber construction over a sufficient period of years to warrant any assumption that can carry much weight; however, it seems safe to assume that maintenance costs will be reduced to a value not more than 50% of the minimum values given in Figure 8 for untreated timber. (See Groups B and D). This assertion is based upon the known fact that first-class pressure treated timber sticks under the worst possible conditions as regards stimulation of decay, will last 25 years to 30 years and even longer, which is more than twice the average life of untreated timber.

If we adopt as a basis for our economic study, the hypothesis that the average resulting maintenance costs in the case of treated timber will be 50% of that for untreated material under the most favorable conditions as regards service life, a curve such as shown in Figure 15 may be drawn for pressure treated timber trestle construction. The combined curve of Figure 15 is constructed over the renewal curve R as a base by simply using as maintenance ordinates 50% of the va... indicated in Figure 8 for the untreated timber trestles of Group B.

Figure 15 indicates a study of trestle construction only; obviously such a study may be extended to include the unhoused truss group if desired. From a study of Figure 15, it will be seen that for the assumed conditions as regards maintenance costs, treated timber trestles will have an economic service life of from 22 to 28 years, as against 11 years for untreated construction under the most unfavorable conditions, or 19 years for untreated construction under the most favorable conditions as regards service life.

Using these figures as a basis for our comparison, and assuming the most unfavorable conditions as regards decay, the economic equations as given by Figure 11 can be compared with a similar equation derived from Figure 15 as follows:

$$\text{Total annual cost} = C\left[r + \frac{M}{c} + \frac{R}{c} + \frac{I}{c}\right]$$

(Neglecting operation costs and rental values which may be done since the riding qualities may be said to balance and the rental value does not properly enter into the problem when balancing two different types of trestle construction.)

Assume $r = 5\%$

For the untreated construction (see Figure 11):

$$\frac{M}{c} = 3.7\%$$

$$\frac{R}{c} = 8.5\%$$

$$\frac{I}{c} = 0.2\%$$

$$\text{Total annual cost} = C\left[r + \frac{M}{c} + \frac{R}{c} + \frac{I}{c}\right] = C\ (17.4\%)$$

For the treated construction (see Figure 15):

$$\frac{R}{c} + \frac{M}{c} = 5.2\%$$

$$\frac{I}{c} = 0.3\%$$ (Assumed at 150% of the value for untreated timber)

$$\text{Total annual cost} = C\left[r + \frac{M}{c} + \frac{R}{c} + \frac{I}{c}\right] = C\ (10.5\%)$$

For *equal economy*, therefore, the treated trestle construction must cost $\frac{17.4}{10.5}$ or 165% of the untreated construction. If it costs less than this, the treated timber is the economical selection and vice versa.

As an example, consider a trestle span designed for Class A loading, 20-foot roadway and composed of 19-foot spans.

*The component cost and quantity items may be taken directly from Figure 8 of Chapter 4, as follows:

ITEM	QUANTITY PER LIN. FT.
Labor	6.7 man-hours
Nails and bolts	14.8 lb.
Paint and preservative	0.10 gals.
Lumber	239.00 ft. B. M.

Assume the average rate for labor for bridge carpenters at 70 cts. per hour, nails and bolts at 10 cts. per pound, paint at $2.00 per gallon and lumber as follows:

Lumber F. O. B. treating plant	$22.00 per M.B.M.
Cost of treatment	28.00 per M.B.M.
Cost of handling at R. R. station	1.00 per M.B.M.
Freight rate on lumber from treating plant, per cwt.	0.30
Length of truck haul from R. R. station to bridge site	10 miles
Cost of truck haul per M.B.M. per mile	0.30
Cost of untreated timber F. O. B. R. R. station nearest bridge site	30.00 per M.B.M.

Then the untreated timber F. O. B. site will cost as follows:

	Per M.B.M.
F. O. B. station	$30.00
Haul to bridge site	3.00
Handling at station	1.00
Total	$34.00

and the treated timber will cost as follows (see Figure 14):

	Per M.B.M.
Cost F. O. B. station	$59.75
Haul to bridge site	3.00
Handling at station	1.00
Total	$64.75

Applying the above unit prices, we have the following comparison:

ITEM	UNTREATED TRESTLE			TREATED TRESTLE		
	Amount	Rate	Total	Amount	Rate	Total
Labor	6.7 M.H.	$0.70	$4.69	6.7 M.H.	$0.70	$4.69
Nails and bolts	14.8 lb.	0.10	1.48	14.8 lb.	0.10	1.48
Paint and preservative	0.10 gal.	2.00	0.20	0.10 gal.	2.00	0.20
Lumber F. O. B. site	239 ft. B.M.	34.00	8.13	239 Ft. B.M.	64.75	15.52
Total			$14.50			$21.89

*In order to simplify the problem, only the superstructure has been considered; obviously a similar comparison may be made of the substructures if desired.

For equivalent economy, the treated construction should cost 165% of $14.50, or $23.93 per lineal foot and since it costs less than this amount, the economics are in favor of treated timber construction.

If, however, it were possible to secure first quality untreated timber, F. O. B. the R. R. station nearest the bridge site for $22.00 per M.B.M. the cost of the untreated trestle would be reduced to the following value per foot of superstructure:

Labor (as before)	$4.69
Nails and bolts (as before)	1.48
Paint and preservative	0.20
Lumber F. O. B. site = 239 ft. B. M. at $26.00 per M	6.21
Total	$12.58

12.58 x 165% = $20.75 per lineal foot, which, in this case, is less than the cost of treated timber construction, so that for these prices there is no economy in adopting a pressure treated material for the trestle superstructure.

The above will serve to illustrate the general nature of the problem and will also serve to indicate that the propriety of using treated timber in any given case is not a clear cut matter, by any means, but is one involving rather closely balanced economies and dependent upon local prices, hauling costs, distance from plant, freight rates, etc.

Each individual condition is susceptible of analysis and should be analysed after having determined all of the pertinent data as to hauling costs, freight rates, creosote prices, etc., for the problem in hand.

The maintenance cost curves of Figure 15 are not authentic by any means and should be supplanted by actual maintenance cost data wherever the same are available. In the absence of better information, however, these data furnish at least a semi-rational method of attacking the problem.

So far the discussion has dealt with pressure treated timber construction only, and, as stated hereinabove, there are two other methods of treating timber with creosote, namely:

(a) The brush coat method.

(b) The open tank process.

The brush coat method is simply a temporary expedient and is used as a retardant of decay in new construction and to a considerable extent for maintenance purposes. It is an excellent practice, as far as it goes, but is not a real preservative treatment in the sense that a pressure treatment may be regarded and, therefore, need not be discussed further at this point.

The open tank treatment can best be described by quoting again from the standard specifications of the A. A. S. H. O. as follows:

"Open tank treatment shall consist of a hot bath treatment or a hot and cold bath treatment as may be specified.

Equipment: All tanks used in the open tank process shall be of sufficient size to allow free circulation of the liquid around the largest amount of timber being treated in any operation.

Sufficient liquid shall be maintained in the tanks to completely immerse the timber. When a number of pieces are being treated at each operation, each stick shall be separated from the others on all sides by square or round spacers not less than ¼ inch in least dimension. Suitable slings and handling devices shall be provided to make the material transfers necessary during the complete process without disturbing the stacked position of the pieces in the bundle.

For hot bath treatments at least one tank shall be supplied having suitable steam coils or other heating device to keep the liquid at a uniform temperature throughout the tank of not less than 240°F. during the complete process.

For hot and cold bath treatments at least one hot tank shall be supplied as for the hot bath treatment and one cold tank having the same capacity as the hot tank. The cold tank shall be equipped with suitable cold water coils or water jackets so that the temperature of the liquid at the time of immersion of each batch of timber shall be no higher than the surrounding atmospheric temperature.

Preparation of Material. All timber to be treated shall be free from dirt, grease or other foreign matter which will in any way hinder the free penetration of the preservative. Framing shall be done before treatment. Round timber or timber with wane shall have the rough bark and inner bark removed as specified for wood piling.

Time of Immersion. The time of immersion as specified herein is for Southern Yellow Pine, Northern White Cedar, Chestnut, Black Gum, West Coast Hemlock, Red Oak, Lodgepole Pine, Norway Pine and Pondosa Pine. The specified time of immersion shall be increased to 66 2/3 per cent for Southern Cypress, Douglas Fir and Red Fir, Idaho White Pine and Northern White Pine and 100 per cent for White Oak and Eastern, Engelmann and Sitka Spruce.

Single or Hot Bath Treatment. The timber shall be completely immersed in preservative in the hot tank, which shall be maintained at a temperature of 190°F. for seasoned timber and 230°F. for timber not seasoned. A tolerance of 10 degrees in either direction is permissible. For seasoned timber the immersion shall be for a period of not less than 15 minutes for two inch timber with an increase of five minutes in the immersion period for each additional inch in thickness. For timber other than seasoned, the immersion period shall be doubled.

Ordinary Hot and Cold Treatment. The timber shall be completely immersed in preservative in the hot tank, which shall be maintained at a temperature of 190°F. for seasoned timber and 230°F. for timber not seasoned. A tolerance of 10 degrees in either direction is permissible. For seasoned timber the immersion shall be for a period of not less than 15 minutes for two inch timber with an increase of five minutes in the immersion period for each additional inch in thickness. For timber other than seasoned, the immersion period shall be doubled. At the end of this period, the timber shall be removed from the hot tank and immediately immersed in the cold tank. At the time of transfer, the preservative in the cold tank shall have a temperature as low as possible, but in no case, higher than the surrounding air temperature. The timber shall be completely immersed in the cold tank for a period one-half as long as the hot bath.

Successive charges from the hot tank may be placed first in one cold tank and the next in a second cold tank, in order to keep the cold tank temperature as low as possible at the time of immersion. Should the contractor supply a cold tank capable of handling all material and with a cooling system which will keep the temperature at the time of all cold treatments as specified, only one cold tank may be required. Single cold tank equipment shall be subject to the approval of the Engineer.

Heavy Hot and Cold Treatment. The requirements for this treatment are the same as those specified above for the Ordinary Hot and Cold Treatment except that the time of immersion in the cold bath shall be the same as the time of immersion in the hot bath."

The double dip process with hot and cold tanks will, in general, secure a better penetration than the single tank treatment. Figure 16 is a cost curve prepared from actual costs of double dip, open tank treatments done by the writer's maintenance crews during the year 1925. The nature of the treatment was quite similar to that specified for the "heavy hot and cold treatment" hereinabove, although the duration of immersion was varied throughout the work between wide limits.

From this figure and from Figure 14, a comparison of the relative cost of open tank *vs.* pressure treatments can easily be made.

For example, in the problem stated above the cost of 8 lb. pressure treated material F. O. B. the site was found to be $64.75 per M.B.M.

For the double dip, open tank treatment, the following costs obtain:

Cost of lumber untreated F. O. B. station	$30.00
Hauling lumber to bridge site	3.00
Handling at R. R. station	1.00
Total cost of lumber F. O. B. plant at bridge site	$34.00
Cost of creosote F. O. B. station	$19.60
Cost of hauling creosote	0.68
Total cost of creosote F. O. B. plant at bridge site	$20.28
TOTAL cost of lumber and creosote	$54.28

To the above must be added the labor and plant cost which depends upon the number of feet of lumber to be treated for this one single plant set-up.

If 100 M.B.M. are to be treated, the plant cost (as indicated by Figure 16) will amount to about $8.40, making a total cost of $62.68 or only slightly less than the cost of pressure treated material. If only 40 M.B.M. are to be treated, the plant and labor costs amount to $15.60 making a total cost of $69.88 or more than the cost of pressure treated material.

In neither case above should the open tank treatment be used, as this method does not secure the penetration into the heartwood that the pressure treatment does. The open tank process penetrates the sap ring and thereby tends to inhibit decay where decay is most apt to occur. However, there is no appreciable penetration of the heartwood which is in sharp contrast to the pressure treatment method which penetrates the sap ring perfectly and "heels well" in to the heartwood.

All in all, the open tank treatment finds little to recommend it, unless the cost can be kept to from 60% to 70% of the corresponding costs for pressure treated material. This method, therefore, is only worthy of con-

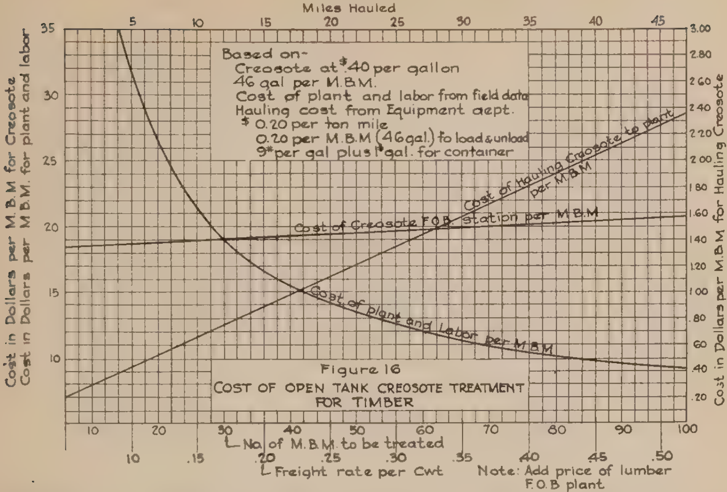


Figure 16
COST OF OPEN TANK CREOSOTE TREATMENT FOR TIMBER

sideration for treating a comparatively large quantity of material in isolated localities far removed from rail transportation facilities and where lumber can be obtained locally at a comparatively low first cost. A few such places exist and where conditions are such as to throw the economy to the side of the open tank method, such method finds a distinct but rather restricted field of utility.

CHAPTER VI

UNIT COSTS

Article 1.—Introductory

The discussion in Chapter IV in reference to first cost data had to do largely with the question of quantities, although some of the curves were plotted in terms of actual costs at varying assumed unit prices. The purpose of the present chapter is to submit data from which to form an estimate regarding unit prices for the various materials entering into the finished construction in order that quantities may be correctly converted into costs.

The principal cost items which enter into the construction of an ordinary highway bridge may be grouped as follows:

1. Dry foundation excavation.
2. Wet foundation excavation.
3. Class A concrete (Beams, girders, arch ribs, etc.)
4. Class B concrete (Piers and abutments).
5. Class D concrete (Floors and thin slabs).
6. Concrete for hydraulic seals.
7. Heavy mass concrete.
8. Stone masonry construction.
9. Concrete handrailings and ballustrades.
10. Metal reinforcement.
11. Structural steel (Ordinary truss construction).
12. Structural steel (Beams and girders).
13. Lumber and timber (For superstructures and decks).
14. Lumber and timber (For trestle substructures and ordinary framing).
15. Lumber and timber (Truss construction).
16. Timber piling.
17. Concrete piling.
18. Miscellaneous and minor items.

19. General equipment and plant costs.
20. General overhead expense.
21. Contractor's profit.

Each of these cost items will be considered in the articles which follow.

Article 2.—Dry Foundation Excavation

This cost item covers ordinary foundation excavation either entirely in the dry or where the water to be contended with is so small in amount as to render any extensive system of cofferdamming unnecessary. Excavation of this kind will cost considerably more than ordinary borrow pit excavation, as the contractor is forced to work in a rather cramped and confined space, and, in general, the quantity of material to be moved is so small as to render it impracticable to properly rig up for the job.

In general the total cost per cubic yard will be made up of the following three component items:

Labor cost.
Materials for braces and shoring.
Equipment charges.

The equipment charge will vary between wide limits, depending upon the size of the job and the nature of the set-up. In certain cases the work will be done almost entirely by hand and no equipment will be used. In other instances, especially where the contractor expects to employ heavy equipment at a later stage in the work, the excavation will be accomplished by means of a clam shell or orange peel bucket, a power hoist or other device and the equipment charge will amount to a considerable percentage of the total cost.

Table I is a compilation of unit costs on seven typical construction jobs recently completed under the writer's direction and serves to indicate the general scope of costs for this class of work.

TABLE I

TABULATION OF UNIT COSTS

DRY FOUNDATION EXCAVATION FOR BRIDGES

Job no.	Cu. yds. involved	Cost per cubic yard				Labor cost, man-hours per cu. yd.
		Labor	Material	Equipment	Total	
A	104	$1.23	$0.24	...	$1.47	No data
B	300	1.76	0.16	$0.10	2.02	No data
C	413	2.22	...	...	2.22	4.32
D	142	2.03	0.26	...	2.29	No data
E	246	1.37	0.94	0.51	2.82	2.20
F	78	3.00	...	0.38	3.38	3.83
G	84	3.35	0.16	0.36	3.87	4.98

The above tabulation discloses a rather startling variation in unit costs and it is, of course, impossible to reconcile the same without a detailed analysis of each individual job which is a procedure too lengthy and much too highly involved to be warranted in a discussion of this kind. Part of the variation may be explained by the character of the material handled, part by the character of the set-up and the contractors organization and part by the weather conditions obtaining during the work and the conditions peculiar to the locality.

The average total cost for ordinary work of this kind as indicated by the above table varies from $2.00 to $2.25 per cubic yard of pay dirt, and the time-labor constant varies from 2 to 5 man-hours per cubic yard excavated.

The above is probably as close as ordinary dry foundation excavation can be estimated without a detailed study of individual conditions and as the pro rata of total cost expended for dry excavation is generally a rather small percentage of the total cost of the structure the above figures are probably close enough for preliminary estimates or comparative cost studies.

There are a number of factors which modify costs of this character and if these can be determined in advance a much closer estimate is possible. A brief discussion of the jobs listed in the above table will be of value in this connection. Job A was a rather large project involving only a very small amount of dry excavation. The work was very well organized and no equipment was needed in addition to that already on th work. Job G, on the other hand, was very poorly organized. The equipment was not chargeable to any other item of work, the time loss due to delays and break-downs was very great and the material to be handled very expensive (heavy gravel and boulders). Job F was well enough organized, but the material was very difficult to handle, consisting of seamy, scab rock and heavy fissure boulders interbedded with mud seams. The amount of excavation removed on Job F was also very small, only sufficient to clean, roughen and serrate the foundations preparatory to the placement of masonry. Work of this character will generally run from $3.00 to $3.50 per cubic yard.

In addition to the above tabulated costs which were for jobs recently completed, the following table gives the resulting total costs for dry excavation, compiled from a few typical construction jobs completed by the writer's organization since 1920.

Job. no.	Total cost per cu. yd. (Dry foundation excavation)
H	$0.75
I	2.08
J	1.43
K	3.07
L	2.06
M	1.95
N	2.06
O	5.63
P	3.24
Q	2.66
R	1.48
S	1.92
T	2.20
U	1.56
V	2.69

Article 3.—Wet Foundation Excavation

The total cost of foundation excavation of this character is generally segregated into two component cost items, as follows:

(a) Cost of cofferdam or crib work.

(b) Labor cost of excavation proper.

It is probably the better policy to segregate these two items and to estimate the cost of cofferdams or cribs on the basis of the superficial areas involved. For timber cribs, the total area of the crib (the perimeter of the crib multiplied by the total height) should be computed. To the resulting area, the following unit prices may be used and although they are only roughly approximate, they are at least close enough for preliminary estimating purposes:

TABLE II

Total height of crib (From foundation to a point 3ft. above working water line)	Cost of crib per. sq. ft. in place
10 ft.	$0.65
20 ft.	0.80
30 ft.	1.00
50 ft.	1.20

The above cost includes the cost of framing, caulking, sand boxes and bracing and also the cost of sinking including the necessary equipment charges.

Where timber sheet piling are used, the cost of cofferdams may be estimated on the basis of the lineal feet of piling driven. The following is a rough approximation of the cost per lineal foot of piling of this character in place.

TABLE III

Type	Cost per lin. ft. in place
Light 6 inch piling	$0.50
Medium, 9 inch piling	0.65
Heavy, 12 inch piling	0.90

Steel sheet piling will vary in cost with the type employed, the haul to the site, the character of driving, etc. Moreover, the sheet piles may be pulled and re-used many times, so that the pro rata cost chargeable to any one job depends upon the amount of loss and depreciation resulting from their use and the probable extent of their re-use. For rough estimates, it is probably safe to charge 25% of the first cost of the piling, plus transportation, plus the cost of handling and driving.

All of the above figures are, of course, only rough approximations, but it must be remembered that we are dealing with unit cost figures applicable to preliminary estimates only and any refinement of analysis at this point would entail the consideration of such a large number of different and varying factors as to entirely defeat the purpose of the discussion. As the job developes and the type selections are made, a much greater refinement is possible for final estimating purposes.

Before concluding the consideration of cofferdam and crib costs, it should be pointed out that the relative cost of crib work, that is to say, the cost of cribs or cofferdams per cubic yard of pay dirt will depend upon the relative depth of water and excavation to be removed. For example, let us consider a crib 30 feet in length, 15 feet in breadth and 30 feet deep. Applying the figures hereinabove given, the total cost of one crib will be as follows (not counting corners):

Total perimeter, $(2 \times 30) + (2 \times 15) = 90$ feet.
Total area, $90 \times 30 = 2700$ sq. ft.
Total cost, $2700 \times \$1.00 = \$2700.00$.

Assume the maximum working water elevation to be 3 feet below the top of the crib. Then for a water depth of 8 feet, the yardage of pay dirt is as follows:

Depth of excavation, $30 - (3+8) = 19$ feet.
Yardage of excavation, $(19 \times 30 \times 15) \div 27 = 316$ cu. yds.

$$\text{Cost per cu. yd.} = \frac{\$2700}{316} = \$8.54$$

If, however, the water were 17 feet deep, the resulting yardage of pay dirt would only be

$$(10 \times 30 \times 15) \div 27 \text{ or } 166 \text{ cu. yds.}$$

and the unit yardage cost for cofferdams would be nearly twice the above figure.

In view of the foregoing, it will be seen that the unit cost per cubic yard of pay dirt for crib or cofferdam work is entirely dependent upon the depth of water over the excavation to be removed.

The second item of cost involved under the heading "wet excavation" is the labor cost of removing the material from the cofferdam, once the same has been sunk to position or driven, as the case may be. The following are actual labor costs, taken from five typical construction jobs under the writer's direction during the years 1926 and 1927.

TABLE IV

Job. No.	Cu. yds. Moved	Labor cost for wet excavation per cu. yd.
A	414	3.14
B	83	5.02
C	487	5.42
D	67	5.48
E	245	7.42

The above costs are all rather high and could probably be cut from 30% to 50% on well organized work of any magnitude.

Article 4.—Class A Concrete

This designation includes the type of concrete ordinarily used for beams, girders, arch ribs and the like. The following extract from the specifications under which the concrete which forms the basis of the cost data given hereinbelow was placed, indicates the general class of material employed:

"For the various concrete mixtures used in the work, the proportions of cement, fine aggregate and coarse aggregate, shall be determined from the strength requirements for the particular class of work in question and from the particular limitations of the aggregate used and the consistency employed in the work.

"As a basis for estimating and as a general indication of the mix employed, the following approximate classification of proportions is given:

Class A Concrete:

1 part Portland cement.

2 parts fine aggregate.

4 parts coarse aggregate passing a two (2) inch opening. (For massive sections or those lightly reinforced, 2½ inch coarse aggregate may be permitted. For concrete which is to be exposed to the action of sea water, the maximum size of coarse aggregate shall be one (1) inch.

Class B Concrete:

1 part Portland cement.

2½ parts fine aggregate.

5 parts coarse aggregate passing a three (3) inch opening.

CLASS D CONCRETE:

(Used for thin reinforced sections and for roadway slabs)

1 part Portland cement.

2 parts fine aggregate.

3 parts coarse aggregate passing a one (1) inch opening.

Where Class "D" concrete is to be used for roadway slabs, the maximum size of aggregate shall be increased to two and one-half (2½) inches.

"The above specified approximate proportions shall be varied as may be directed by the Engineer, to secure from test cylinders the following minimum compressive strengths in pounds per square inch at the ages of seven and 28 days:

CLASS OF CONCRETE	7 DAYS	28 DAYS
A	1300	2200
B	1000	1700
D	1700	2800

"Frequent tests, both in the laboratory and field, shall be made of the concrete mixtures, and the proportions of fine aggregate, coarse aggregate and cement shall be varied in accordance with these tests in order to produce the above specified compressive strengths.

"The laboratory tests made to determine the mortar and concrete making qualities of aggregate shall, in general, involve the use of these materials in a surface dried condition and in the records and reports of tests the quantities of aggregates shall be given for dry materials, either by weight or by volume, or both.

"Laboratory tests shall be made by the method for 'Compression Test of Concrete' (A. S. T. M. Tentative Method, Serial Designation C39-21T, modified) and field specimens shall be made by the 'Method of Making and Storing Specimens of Concrete in the Field' (A. S. T. M. Standard Method, Serial Designation C31-21, slightly modified), both as provided in the Tentative Standard Methods of Sampling and Testing of the American Association of State Highway Officials.

"To form a basis for bidding and for payment on concrete, the cement required for any structure shall be determined from the proportions as above specified for the different classes of concrete employed and the total yardage placed. For this purpose, the following approximate quantities shall furnish a basis:

TABLE OF MATERIALS FOR ONE CUBIC YARD OF COMPACTED CONCRETE

CLASS OF CONCRETE	NOMINAL PROPORTIONS	CEMENT SACKS	WEIGHT OF CEMENT POUNDS
A	1:2 :4	6.3	592
B	1:2½:5	5.2	489
D	1:2 :3	7.3	686

"If the contract price for concrete includes the cost of cement and if the proportions as adopted for the work vary from those specified above, necessitating a change in the quantity of cement, the difference in the cost of the work, based upon the actual cost of the cement delivered at the bridge site, shall be adjusted for or against the contractor, as the case may be.

"Changes in the proportion of fine to coarse aggregate shall not be cause for any adjustment in compensation except as such changes affect the quantity of cement required. However, changes made at the contractor's request to permit the use of inferior aggregates shall not involve additional compensation for cement used."

The various component cost items for Class A concrete are as follows:

Cement
Coarse Aggregate
Fine Aggregate
Forms and Falsework
Mixing and placing
Finishing
Equipment charges

Table V is from seven typical construction jobs recently finished and indicates the average range of costs for this class of work:

TABLE V

TABALATION OF COST ANALYSIS

CLASS A CONCRETE

	Cost per cu. yd.						
Item	Job No.—A	B	C	D	E	F	G
Cement	$5.85	$5.51	$6.06	$5.22	$6.44	$7.09	$7.56
Coarse aggregate	2.02	2.53	2.13	2.08	3.38	2.75	1.41
Fine aggregate	1.60	1.96	1.22	1.27	1.80	1.27	0.81
Forms and falsework:							
Lumber	2.38	1.83	2.41	2.06	1.48	1.52	2.69
Hardware	0.14	0.24	0.23	0.48	0.11	0.10	0.30
Labor	7.47	2.39	4.00	5.98	4.53	3.18	6.75
Mixing and placing	3.08	2.37	3.45	1.55	4.69	1.21	3.43
Finishing	0.72	3.17	1.03	1.09	0.71	0.17	0.60
Equipment charges	0.75	0.97	0.59	1.92	3.47	1.72	...
Total cost per cu. yd.	$24.01	$20.97	$21.13	$21.65	$26.61	$19.21	$23.55
Yardage placed	346	124	532	1141	146	426	319
Labor costs in man-hours per cu. yd.:							
Forms and falsework	...	3.83	7.35	10.14	5.94	...	...
Mixing and placing	...	4.00	6.69	3.12	7.23	...	...
Finishing	...	...	2.01	1.92	0.90	...	...
Cost of forms and falsework per sq. ft. surface of concrete:							
Lumber	$0.053	$0.024	$0.060	$0.060	...	...	...
Hardware	.003	.003	.006	.010	...	...	...
Labor	.173	.031	.110	.160	...	...	...
Cost of finishing:							
Per sq. ft. of surface	$0.016	*$0.086	$0.030	$0.030	...	...	...

*This cost is for special plaster finish and includes cost of special white sand and white cement. Labor cost alone amounts to $0.05 per square foot.

Article 5.—Class B Concrete

This designation includes the type of concrete ordinarily employed

for piers and abutments (see extract from specifications given in the preceding article).

The average unit costs for six typical construction jobs are given in Table VI hereinbelow. Three of these jobs are ones recently completed; the others are somewhat older.

TABLE VI

TABULATION OF COST ANALYSIS

CLASS B CONCRETE

Item	Job. No.—A	B	C	D	E	*F
	Cost per cu. yd.					
Cement	$4.53	$5.98	$4.37	$4.47	$4.17	$5.06
Coarse aggregate	3.58	2.75	1.20	2.83	2.04	1.19
Fine aggregate	0.84	1.67	0.90	2.53	1.10	0.70
Forms and falsework:						
Lumber (F O. B. job)	2.85	3.22	1.02	1.37	0.68	0.87
Hardware (F. O. B. job)	0.28	0.20	0.14			
Labor	3.94	4.67	3.54	3.90	1.60	2.66
Mixing and placing	2.97	3.41	2.20	1.30	3.63	2.88
Finishing	1.47	0.57	0.50	0.40	...	0.19
Equipment charges	2.92	...	0.30	1.22	...	0.19
Total cost per cu. yd.	$23.38	$22.47	$14.17	$17.82	$16.09	$13.74
Total yardage placed	293	186	303	205	102	311

*Does not include cost of hauling materials to site.

Article 6.—Class D Concrete

This designation includes the type of concrete ordinarily employed for floors and thin slabs. The following table (Table VII) is compiled from the cost records on six typical construction jobs recently completed under the writer's direction:

TABLE VII

TABULATION OF COST ANALYSIS

CLASS D CONCRETE

Item	Job. No.—A	B	C	D	E	F
	Cost per cu. yd.					
Cement	$7.46	$8.82	$6.58	$4.97	$6.58	$6.53
Coarse aggregate	2.12	3.31	2.53	1.98	2.04	3.83
Fine aggregate	1.22	1.76	1.97	1.34	1.44	0.53
Forms and falsework:						
Lumber (F. O. B. job)	2.27	1.74	2.09	4.76	1.65	4.48
Hardware (F. O. B. job)	0.19	0.10	0.13	0.63	0.15	0.41
Labor	4.82	4.19	2.34	8.55	5.57	3.92
Mixing and placing	4.30	3.31	1.58	1.09	3.29	3.52
Finishing	0.55	0.95	(Incl. above)	0.13	0.54	0.38
Equipment charges	...	2.73	0.54	2.68	0.53	2.95

TABLE VII (Continued)

ITEM	JOB. NO.—A	B	C	D	E	F
	COST PER CU. YD.					
Total cost per cu. yd.	$22.93	$26.91	$17.76	$26.13	$21.79	$26.55
Yardage placed	111.0	71.0	83	314	138	196
LABOR COST IN MAN-HOURS PER CU. YD.:						
Forms and falsework	8.87	5.90	2.76	13.20	...	5.00
Mixing and placing	8.32	4.98	2.67	2.99	...	4.45
Finishing	1.08	1.46	...	0.24	...	0.49
COST OF FORM WORK PER SQ. FT. OF SURFACE:						
Lumber	$0.007	$0.033	$0.063	$0.120	$0.05	$0.08
Hardware	0.007	0.002	0.007	0.020	0.005	0.008
Labor	0.015	0.081	0.070	0.220	0.172	0.073
COST OF FINISHING PER SQ. FT. OF SURFACE:						
Finishing	$0.021	$0.022	....	$0.003	$0.016	$0.07

ARTICLE 7.—CONCRETE FOR HYDRAULIC SEALS

Concrete of this classification is for placement as a seal for the bottom of a cofferdam or crib. Such concrete must be placed under water and generally a tremie or bottom dump bucket is used. The following extract from specifications governing this class of work indicates the general methods involved and the nature of the material:

"All concrete deposited under water shall be mixed in the proportions designated for Class "A" concrete with 10 per cent excess cement added.

"Concrete deposited under water shall be carefully placed in a compacted mass in its final position by means of a tremie, a closed bottom dump bucket or other approved method and shall not be disturbed after being deposited. Special care must be exercised to maintain still water at the point of deposit. No concrete shall be placed in running water and all form work designed to retain concrete under water shall be water-tight. The consistency of the concrete shall be carefully regulated and special care shall be exercised to prevent segregation of the materials. The method of depositing concrete shall be so regulated as to produce approximately horizontal surfaces.

"When a tremie is used, it shall consist of a tube having a diameter of not less than 10 inches, constructed in sections having flanged couplings fitted with gaskets. The means of supporting the tremie shall be such as to permit the free movement of the discharge end over the entire top surface of the work and shall be such as to permit it to be rapidly lowered when necessary to choke off or retard the flow. The discharge end shall be entirely sealed at all times and the tremie tube kept full to the bottom of the hopper. When a batch is dumped into the hopper the tremie shall be slightly raised (but not out of the concrete at the bottom), until the batch discharges to the bottom of the hopper. The flow is then stopped by lowering the tremie. The flow shall be continuous and in no case shall it be interrupted until the work is complete.

"When concrete is placed by means of a bottom dump bucket, the bucket shall have a capacity of not less than ½ cu. yd. The bucket shall be lowered gradually and carefully until it rests upon the concrete already placed. It shall then be raised very slowly during the discharge travel, the intent being to maintain,

as nearly as possible, still water at the point of discharge and to avoid agitating the mixture."

It will be observed that the amount of cement per cubic yard is greater than for any other class of concrete, while the cost of form work and for finish is eliminated entirely. A recent job completed by the writer ran as follows:

Item	Cost per cu. yd.
Cement	$7.80
Coarse aggregate	2.75
Fine aggregate	1.45
Mixing and placing	1.77
Equipment charge (not included)	...
	$12.77

The writer does not have any other recent cost data from his own work which is properly segregated to admit of insertion at this point but a rough average cost may be obtained from the unit costs for Class A concrete by increasing the cement cost by 10% and omitting the costs for forms and falsework and for finishing.

Article 8.—Heavy Mass Concrete and Stone Masonry Construction

Mass concrete work will vary in cost from $11.00 to $16.00 per cubic yard, depending upon conditions. The writer has done very little of such work during the past few years. It may be said in general that a rough indication of unit costs may be obtained from the data given in Table VI for Class B concrete, by multiplying those unit costs by the following coefficients:

Item	Coefficient
Cement	90%
Coarse aggregate	100%
Fine aggregate	100%
Forms and falsework	40%
Mixing and placing	60%
Finishing	10%
Equipment charges	100%

Stone masonry work will vary in cost between wide limits, depending upon local conditions.

Ordinary rubble masonry for bridge abutments will cost about the same as mass concrete where suitable building stone may be obtained locally at a reasonably short haul. Coursed rubble work will cost somewhat more than random rubble and cut and tooled stone work will run much higher in cost than any concrete work.

The range in price for stone masonry work is so great and the

variation so dependent upon the specifications employed, the accessibility of suitable material, the character and shape of the construction contemplated, etc., that any attempt to present average unit cost data might easily prove misleading.

Under certain conditions, stone masonry will show an economy in first cost over mass concrete, but these are local conditions and must be studied locally.

Article 9.—Concrete Handrailings and Balustrades

The cost of this work will vary with the type of rail adopted, with the cost of materials at the job site and with the skill of the construction crew. The following tabulation is from the writer's cost records covering a large number of jobs of the general type shown in Chapter IV. A more ornamental type of rail will, of course, run higher.

TABLE VIII

UNIT COSTS PER LINEAL FOOT OF RAIL
CONCRETE HANDRAILS AND BALUSTRADES

		Unit costs per lineal foot			
Job No.	No. of lin. ft. of rail	Concrete and form materials	Reinforcing	Labor	Total
A	464	1.72	Included in Materials	1.50	3.22
B	250	2.47		0.31	3.32
C	900	1.51		0.93	2.44
D	434	1.60		1.40	3.00
E	462	1.75	0.36	0.65	2.76
F	300	0.79	0.26	2.29	3.34
G	301	0.79	0.19	1.72	2.70
H	860	0.57	0.16	1.90	2.63
I	183	0.41	0.17	2.19	2.77
J	248	0.41	0.23	2.35	2.99
K	54	0.65	0.31	1.30	2.26
L	51	0.64	0.32	1.30	2.25
M	71	0.83	0.27	1.30	2.40
N	658	0.84	...	3.04	3.88
O	50	0.48	0.23	2.52	3.33
P	222	1.31	0.28	4.14	5.73
Q	210	2.50	0.25	1.00	3.75
R	92	1.35	0.54	2.95	4.84

Article 10.—Metal Reinforcement

The cost of material F. O. B. job is purely one of securing current prices and compiling hauling costs and need not be given here. The labor costs of placing on nine typical construction jobs recently completed by the writer are as follows:

Job No.	Cost of placing reinforcing steel (per 100 lb.)
A	$0.41
B	0.50
C	0.58
D	0.70
E	0.83
F	0.90
G	0.96
H	1.17
I	1.20

Article 11.—Structural Steel

For ordinary truss construction, the average unit costs are indicated with reasonable accuracy by the following tabulation from five typical construction jobs recently completed:

TABLE IX

TABULATION OF COST ANALYSES

STRUCTURAL STEEL IN PLACE
ORDINARY HIGHWAY BRIDGE TRUSSES

	Unit cost per pound				
Item	Job No.—A	B	C	D	E
Material F. O. B. job	$.06000	.05025	.05500	.05900	.05600
Falsework material	.00400	.00043	.00180	.00200	.00400
Labor on falsework	.00200	.00090	.00300	.00110	.00100
Erecting	.00400	.01104	.00360	.00560	.00300
Riveting	.00400	.00788	.00580	.00580	.00500
Field painting:					
Material	.00100	.00078	.00090	.00080	.00100
Labor	.00100	.00136	.00150	.00160	.00200
Equipment charges	.00100	.00231	.....	.....	.01000
Total cost per pound erected	.07700	.07495	.07160	.07590	.08200

The above costs are for truss construction. Plate girder work and rolled beams will probably erect for about 80% of the above.

Article 12.—Lumber and Timber

This designation includes three distinct classes of work which vary greatly in cost as follows:

(a) Timber substructures
(Posts, caps, sills and braces, etc.)
(b) Lumber and timber in superstructures
(Decking, handrails, stringers, etc.)
(c) Timber truss construction

Substructure Construction.—The cost of lumber and timber for substructures will vary with the cost of lumber, with the hauling costs, with the general character of the locality as regards labor costs and also with the height of the trestle work. The following tabulation of costs is from four typical substructure jobs recently completed and may be taken as representative of average conditions in 1927 and 1928:

TABLE X

TABULATION OF COST ANALYSES

LUMBER AND TIMBER FOR SUBSTRUCTURES

	Unit cost per thousand ft. B.M.			
Item	Job No.—A	B	C	D
Lumber (F. O. B. job)	31.00	20.17	25.00	32.00
Hardware	4.23	6.13	4.00	6.70
Labor costs	26.73	27.49	21.25	22.16
Preservatives	2.04	....	**11.75	....
Equipment charges	*....	3.50	*....	10.60
Total	64.00	57.29	62.00	71.46

Notes: *Included in labor costs.
**Double dip open tank treatment on ground.

The variation in lumber prices F. O. B. the job is quite marked in the above table. This variation is due to the difference in hauling costs and the general accessibility of lumber mills or yards. The labor costs are fairly uniform, averaging around $24.00 per thousand. On two of these jobs the equipment charges were small and were "buried" in the general labor cost. On Job D the heavy equipment charge was due to the necessity for very expensive equipment for handling other portions of the work. This equipment was idle a great deal of the time and in rendering a cost analysis for the entire job a portion of this idle time was pro-rated against the lumber and timber for substructures. Such a charge, however, is unusual and may be considered as a special charge and not indicative. Equipment charges for this class of work generally consist of the rental costs of derricks and falls for handling the timber and should not run above $3.00 to $5.00 per M. For low frame structures, the bents may be raised and the timbers handled by hand, thus reducing the equipment charge to a negligible quantity. This was the case in Jobs A and C above.

The cost of paint and preservatives covers the cost of treating ends and contact surfaces of posts and sills with creosote oil and will not run to exceed $1.50 to $2.50 per M. No preservative was used on Jobs B and D, but on Job C all timbers were treated by complete immersion in

a hot tank of creosote oil, followed by a cold tank dip, as described in Chapter IV.

Superstructure Construction.—Labor for superstructure will generally cost less per thousand than substructure work, as the amount of material handled is greater per operation. Stringers and decking go together rather rapidly and no special equipment is needed. The costs for lumber F. O. B. job and for paint and preservatives on the other hand are generally somewhat greater than for substructure work.

The following is a tabulation of the superstructure costs on the four jobs listed above in Table X. A comparison of the unit costs given hereinbelow with the corresponding unit cost for substructure work on the same job, will illustrate the foregoing remarks:

TABLE XI

TABULATION OF COST ANALYSES

LUMBER AND TIMBER FOR SUPERSTRUCTURES

Item	Unit cost per thousand ft. B.M. Job No.—A	B	C	D
Material F. O. B. job	34.20	21.50	25.00	32.00
Hardware	1.99	2.72	4.00	3.18
Labor costs	13.80	8.92	13.50	8.50
Paint and preservatives	2.78	2.00	12.09	2.18
Equipment costs	...	...	...	6.63
Total	52.77	35.14	54.59	54.49

As in the case of the substructure table (Table X) it should be explained that the item of $12.09 per M for paint and preservatives for Job C includes a complete immersion treatment of all decking and stringers. Also the equipment charge of $6.63 per M for Job D covers a pro rata of lost time on general equipment and is not indicative of general costs.

Job B was an exceptionally cheap job, representing a rather unusual case, the superstructure work on this job consisted of several thousand feet of approach and also decking and stringers on a steel span. The crew was well organized, the job was right at the outskirts of a fair sized town, the crew was small and the foreman worked himself. It will also be noted that a very low price for lumber was secured, owing to the fact that the job was immediately adjacent to a large saw mill. The low costs for labor combined with this low lumber cost resulted in a total unit cost much below the average to be expected.

Timber Truss Construction.—This class of work will cost considerably more than the classes of work hereinabove listed, owing to the increased labor costs for falsework and framing and to the fact that the

quality of lumber required and the size and length of sticks render it necessary to purchase rather expensive material.

Unfortunately, the only cost data the writer has for this class of work dates back several years, during a time when lumber and labor prices were considerably higher than at present (1927-1928) and therefore such data are not truly comparable with the data given in Tables X and XI above.

It is probable that with prices comparable to those given in Tables X and XI, the costs for timber truss construction would average about as indicated hereinbelow.

TABLE XII

AVERAGE UNIT COSTS FOR TIMBER TRUSS SUPERSTRUCTURES

Item	Average unit cost per M ft. B.M.
Material F. O. B. job	$35.00
Hardware (not including castings)	6.00
Labor costs (including falsework)	37.50
Paint and preservative	6.50
Equipment costs	4.00
Total	$88.00

It must be noted in passing that the prices for lumber and timber F. O. B. the work will vary widely between different jobs and with different localities and for this reason the cost data given in the foregoing tables will need modification to conform to other existing prices and conditions. For this reason perhaps the following summarization (excluding the cost of lumber and hauling) will be of service in preparing preliminary estimates when the cost of materials F. O. B. job is known or can be determined:

Class of work	Average unit cost per thousand ft.				
	Labor	Hardware	Paint and preservatives	Equip.	Total
Timber substructures	$24.00	$5.00	$1.00	$2.50	$32.50
Timber superstructures	12.00	2.50	2.00	1.00	17.50
Timber trusses	37.50	6.00	6.50	4.00	54.00

Article 13.—Piling

The cost of timber piling F. O. B. the site of the work will vary between wide limits, depending upon the accessibility of suitable material. The writer has seen very good local piling delivered by water at the site for 8 cts. per lineal foot; on the other hand, where piling must be shipped, rehandled and truck hauled to the site, the cost may run as high as 60 cts. to 75 cts. per foot F. O. B. the work.

The cost of driving will, of course, vary with the character of material

to be penetrated, the number of piles to be driven and the equipment needed.

It should be borne in mind that one large expense item, viz.: moving driver in, setting up same, dismantling same at close of work and moving out constitutes a fixed charge independent of the number of lineal feet of piling driven and therefore the cost of driving depends to a large extent upon the magnitude of the work.

On two jobs recently completed, the cost of driving timber piles was as follows in cents per foot:

Job	Labor cost	Equipment charge	No. of ft. driven
A	.16	.02½	6654
B	.27	.11	600

These two jobs illustrate the variation in costs for equipment with the number of feet to be driven.

For preliminary estimating purposes the following unit values may be assumed for ordinary conditions without material error:

No. of lin. ft. to be driven	Cost per lin. ft. (Labor and equipment) Water driving	Land driving
500	45c	55c
2000	35c	43c
5000	25c	30c
10000	20c	23c

The cost of concrete piles will depend upon the design, the character of the driving, the penetration, etc. On a recent construction job completed under the writer's direction the itemized unit costs for driving 2,047 lin. ft. of precast concrete piles, octagonal in shape and driven to an average penetration of about 35 feet through rather difficult driving and under rather adverse conditions, was as follows.

Item	Cost per lin. ft. of pile
Form lumber	$0.97
Labor on forms	.362
Hardware	.010
Reinforcing metal	.044
Cement	.530
Sand and gravel	.210
Mixing and placing	.130
Driving and	.840
Miscellaneous	.180
	$2.403

These piling were 18 inches in diameter and cast of 1 :2 :3 concrete.

Under ordinary conditions piling of this size can undoubtedly be made and driven at a cost of from $2.00 to $2.25 per lineal foot. For other sized piles, the cost of casting and driving will be substantially the same, the only variation being in the amount of materials used.

Article 14.—Miscellaneous and Minor Items

The foregoing paragraphs have treated the individual items of expense which, in general, go to make up the total cost of labor and materials for the ordinary highway bridge job. In addition to these there are always a number of minor expense items which must be charged somewhere. As an illustration, the following is taken from a cost analysis report on a job recently completed:

Total cost of job..$18,373.88

Total yardage of concrete

Class A................346 c.y.

Class D................138 c.y.

Minor expense items

Item	Cost	Cost per c.y. concrete Class A	Cost per c.y. concrete Class D
Buildings and bunkers	$280.83	.59c	.59c
Bronze exp. plates	252.06	.73c	..
Expansion joints and water stops	292.21	..	2.11c

Many times these items are overlooked in estimating and many times it is rather difficult to predict or anticipate certain items of minor expense, for which reason it is a safe policy and one the wisdom of which is amply borne out in practice, to add an arbitrary three (3) per cent to the naked labor and material costs to cover items of miscellaneous and minor expense.

Article 15.—General Equipment and Plant Costs

This designation includes the cost of the general construction plant, camp set up where necessary and all equipment costs not definitely chargeable to any one segregated operation or cost item.

As indicating the general range in expense of this character, the following tabulation from seven construction jobs recently completed will prove of value:

TABLE XIII

TABULATION OF GENERAL EQUIPMENT AND PLANT COSTS

Job No.—	A	B	C	D	E	F	G
Total cost of labor, plus materials	$21006	$21666	$6991	$46400	$70319	$87562	$27945
General equipment and plant cost	1388	302	118	1645	8677	8973	953
Percentage of labor and material costs expended for plant and general equipment	6.6%	1.4%	1.6%	3.5%	12.3%	10.2%	3.4%

As illustrating the general scope of items included in the above classification, the following is a tabulation of the items of expense that go to make up the total of $8,973 shown under Job F hereinabove:

Freight on plant	$2462.05
Miscellaneous hauling	72.62
Tools and hardware for plant	1000.55
Gas and oil	650.00
Fuel oil and wood	1314.62
Derrick and pile driver	993.99
Rowboat and rental on scow	110.70
Plant maintenance, including blacksmith work	1069.06
Washing gravel (including plant)	149.38
Truck maintenance and repair	350.95
Blocks and lines (depreciation)	230.00
Labor shipping equipment	120.90
Installing oil burner in donkey engine	37.13
Industrial railroad for pouring concrete	256.30
Air plant	154.75
Total	$8973.00

Article 16.—General Overhead Expense

In addition to the general equipment and plant costs there will be a general overhead expense for such items as bond, insurance, office maintenance, etc.

As indicative of the general range of expense of this character the following tabulation from nine construction jobs recently completed is submitted:

TABLE XIV

TABULATION OF GENERAL OVERHEAD EXPENSE

Job No.—	A	B	C	D	E	F	G	H	I
Total cost (labor plus materials)	17455	21006	21266	6991	20146	46400	70319	87562	27945
General overhead expense	918	2895	974	247	3130	2324	3430	8792	2114
Percentage of (labor plus materials) cost	5.2%	13.8%	4.6%	3.5%	15.4%	5.0%	4.8%	10%	7.5%

As illustrating the general scope of items included in the above classification, a tabulation of the items of expense that go to make up the total of $8,792 for the above Job H is given hereinbelow, as follows.

Superintendent	$3084.50
Superintendent's car	320.00
Time keeper	575.67
Transportation of men	137.35
Tool house and cement shed	258.63
Gas and oil	650.00

Travel expense	182.62
Telephone and telegraph	52.71
Clearing up bridge site	258.85
Bond	1564.80
Industrial accident insurance	1523.80
Headquarters office (labor)	79.81
Flood protection during const.	15.00
Watchmen	20.00
Clearing right-of-way	12.01
Material yard	56.25
Total	$8792.00

Article 17.—Contractor's Profit

From the foregoing discussion it may be concluded that for estimating purposes the naked cost of labor plus materials should be augmented by adding an arbitrary three (3) per cent to cover the cost of miscellaneous and minor items; also, from 5% to 10% to cover the general cost of plant and equipment and also an increase of about 7% to cover items of general overhead expense.

To the grand total increased as above provided, should be added the contractor's profit—not less than 10%, if reliable firms are to be attracted to the work.

At first glance it may perhaps seem that this last item is an unnecessary expense and that the state or municipality could well afford to eliminate the same by engaging in the work itself. In this connection it must be remembered that the costs we are using and have used hereinabove are contractor's costs—costs by organized agencies—properly equipped to engage in just such an undertaking and whenever the government or any municipality undertakes to engage in such a business, it must expect either to equip and organize on an extensive scale or else to be content with unit construction costs much higher than those given throughout this chapter. On the other hand, the contractor must (if he would justify his own existence) be so organized as to do work at cost plus profit as cheap or cheaper than the actual cost alone of state or municipal force account work. In other words, the contractor must earn his profit to justify it and he usually does since contract costs, including profit, are generally lower than corresponding costs by state or municipal forces.

With the above understanding, therefore, and bearing in mind that the unit costs given throughout this chapter are contractor's unit costs, *the item for profit constitutes a legitimate charge.*

ARTICLE 18.—DETAILED COST REPORTS

In order to illustrate the general scope of cost analysis reports which furnish the basis for data such as have been given hereinabove, it may be of interest to include two or three typical reports of this character.

Report No. 1 was submitted by Mr. Glenn S. Paxson, Assistant State Bridge Engineer and formerly general superintendent of bridge construction by state forces in Oregon. This report covers the construction of a short timber deck truss span and although the cost data refer to work done several years back the report is of value not only for indicating the general arrangement of data needed, but by substituting present prices and present wages these data are as good today as when taken. A radical departure in type of construction equipment is about the only thing that will render these figures invaluable. This report is as follows:

REPORT NO. 1

DETAILED COST REPORT

STATE FORCE BRIDGE CONSTRUCTION
Bridge No. 537

GENERAL DATA:

This bridge was built across Dry Creek, a small tributary of Crooked River about seven miles from Prineville on the Crooked River Highway. This creek is normally dry, except during the break up in the spring when melting snow brings down from 4 to 8 feet of water. The territory drained is subject to cloud-bursts during May and June, which have resulted in flood waters higher than the normal spring flow. The foundations for the piers were placed 4 feet below stream bed and rested upon gravel mixed with large boulders. The excavation was begun before the spring break up and completed after the surface water had run off. A large flow of water was encountered about two feet below the surface. Considerable trouble was experienced with pumping equipment which ran the excavation costs rather high. The material handled was difficult. All of it had to be picked, and a large number of boulders were found that had to be broken up before they could be removed. There was a total of 100 cu. yds. moved in 1057 man-hours or 10.57 man-hours per cu. yd. This includes the time spent repairing and operating the pump.

Sand and gravel from a bar in Crooked River two miles from the bridge was used. It was screened and reproportioned before mixing. The sand and gravel was hauled by team. The labor of screening and loading amounted to 278 man-hours or 3.47 man-hours per cu. yd. for the 80 cu. yds. used. It was hauled in 140 team-hours or 1.75 team-hours per cu. yd.

The forms for the barrels of the piers were cut at a mill in Prineville and were assembled outside the excavation and then set in place. The labor on the forms amounted to 195 man-hours or 3.55 man-hours per cu. yd. for the 55.03 cu. yds. of concrete placed.

A large steam paving mixer belonging to the Highway Department was used, owing to the fact that no other equipment was available at the time. Due

to its excessive weight and to the condition of the roads, the cost of getting this mixer onto the job and setting it up was high. The mixing and placing was done in 196 man-hours or 3.56 man-hours per cu. yd.

There was one 19 foot approach bent on each end of the truss. The total lumber in these two bents amounted to 8139 F.B.M. It was framed in 64 man-hours or 7.86 man-hours per M.B.M. It was erected in 204 man-hours or 25.06 man-hours per M.B.M. The painting took 20 man-hours.

The truss was a 40-foot pony span with the deck superelevated 6 inches for a nine degree curve. The total lumber in the truss was 11,045 F.B.M. This was framed in 153 man-hours or 13.85 man-hours per M.B.M. The truss was erected in 357 man-hours or 32.32 man-hours per M.B.M. The painting was done in 82 man-hours.

Very little false work was needed for this short a span. One bent was erected at the center of the truss. The bottom chords were placed and the deck laid before the remainder of the truss was put up. As the floor beams set on top of the chord, the deck came high enough to allow working on the truss without scaffolding. Some trouble was encountered with the superelevation shims for the truss. These were sent out from the mill as 10x12s, each stick to make two shims. The cost of ripping these timbers by hand with the equipment on the job would have been high, for which reason these shims were taken to a small mill near Prineville and cut there.

The materials for the bridge were hauled by contract. The price on lumber being $0.50 per M.B.M. per mile and on cement and steel $0.40 per ton per mile.

The contractor doing the grading on the Crooked River Highway had completed his work before the bridge was commenced and the approach back fill was made by day labor. About 65 cu. yds. of material were used at a cost of $0.46 per cu. yd.

Construction was begun on January 18th but had to shut down February 18th on account of high water in Dry Creek. Work began again April 4th and the bridge was completed May 5th. The wages paid were $4.50 for laborers and $7.00 for carpenters up to April 1st, and $3.50 and $4.00 for laborers and $6.00 for carpenters after that time. Teams with drivers received $6.00 per day.

The detailed costs are as follows:

I—GENERAL EQUIPMENT COSTS:		
Inventory value of equipment transferred to this bridge	$237.10	
Equipment purchased	45.03	
Rentals paid out for equipment rented	30.00	
Total	312.13	
Less inventory value of equipment transferred from this bridge at end of job	218.75	
Net general equipment charge		$ 93.88
II—ENGINEERING, SUPERVISION AND MISCELLANEOUS OVERHEAD EXPENSE:		
Expense, Ford touring car No. 40	114.23	
Expense, Ford touring car No. 46	101.94	
Salary of superintendent	304.37	
Truck No. 59 transporting men to and from work	95.10	
Miscellaneous minor expense items	118.84	
Total cost of eng'r, super., etc.		734.48
Total equipment plus overhead (I+II)		$827.86

DISTRIBUTION OF ITEMS I AND II:

Excavation	$134.16	
Concrete (Class B)	225.21	
Steel reinforcing	18.13	
Trestle approach	145.13	
Truss span	305.23	
	$827.86	

III—EXCAVATION:

Labor, 1057 man-hours		$634.45	
Rental on pump and engine		11.25	
Repairs to pump and engine		15.78	
Gas—15 gals. at 33½c		5.03	
Supplies		11.60	
Miscellaneous (see distribution under II)		134.16	
Total cost of excavation			$ 812.27
Yardage moved ......100 cu. yds.			
Unit cost per cu. yd. $8.12			

IV—CLASS B CONCRETE (Piers):

(1) SAND AND GRAVEL:

Building road to gravel bar—20 man-hours		$ 13.50	
Screening sand and gravel, 278 man-hours		173.26	
Hauling sand and gravel, 140 team-hours		120.50	
Total			$ 307.26
Unit cost per cu. yd. of gravel and sand		$ 3.84	
Unit cost per cu. yd. of concrete placed		5.58	

(2) CEMENT:

55 bbls. at $3.00 F. O. B. Gold Hill		165.00	
Freight 55 bbls. at $1.94 per bbl		106.70	
Hauling, $0.40 per ton per mile		27.20	
Total cost cement			$ 298.90
55.03 cu. yds. concrete poured:			
Unit cost per bbl	$ 5.43		
Unit cost per cu. yd. of concrete placed	$ 5.43		

(3) FORMS:

Lumber 513 F.B.M. at $33.75	$ 17.31		
1664 F.B.M. at 31.50	52.42		
1477 F.B.M. at 27.00	39.88		
600 F.B.M. at 25.20	15.12		
	$124.73	124.73	
Hauling 4254 F.B.M. at .50 per M per mile		17.20	
Nails		.90	
Labor, 195 man-hours		157.52	
5 hours band-saw work at $3.00		15.00	
Total cost forms			$ 315.17
55.03 cu. yds. concrete poured:			
Unit cost per cu. yd. of concrete placed		$ 5.73	

(4) MIXING AND PLACING:

Rental on pump and engine	13.25		
Pipe fittings	1.60		
Labor, 196 man-hours	104.79		
Total cost mixing and placing			$ 119.64
55.03 cu. yds. concrete poured:			
Unit cost per cu. yd. concrete placed	$ 2.17		

(5) CONCRETE PLANT:			
Hauling mixer to job		32.37	
Supplies		10.31	
Labor setting up runways, etc., 74 man-hours		46.49	
Total cost concrete plant			89.17
55.03 cu. yds. poured:			
Unit cost per cu. yd. of concrete placed	$ 1.62		
(6) FINISHING CONCRETE:			
Labor, 16 man-hours		8.00	
Total cost finishing concrete			8.00
55.03 cu. yds. concrete poured:			
Unit cost per cu. yd. of concrete placed	$ 0.14		
(7) MISCELLANEOUS:			
(See distribution of equipment, overhead, etc., above)		225.21	
55.03 cu. yds. concrete poured:			
Unit cost per cu. yd. of concrete placed	$ 4.09		
			$1363.35

Total unit cost for concrete $24.78

V—REINFORCING STEEL:			
807 lb. ½″ sq. at .056 per lb.		$ 45.19	
443 lb. 1″ sq. at .055 per lb.		24.36	
Freight		11.81	
Hauling, at $0.40 per ton per mile		4.26	
Placing, 8 man-hours		6.00	
Misc. (see distribution under II above)		18.13	
Total cost reinforcing steel			$ 109.75
1250 lb. placed:			

Unit cost per lb. $0.088

VI—TRESTLE:			
(1) 8139 F.B.M. LUMBER AT $41.00 PER M	333.70		
Freight	110.23		
Hauling at .50 per M per mile	45.05		
Total			488.98
8139 F.B.M.			
Unit cost per M at site	$ 60.08		
(2) PAINT AND CARBOLINEUM		9.00	
Freight		3.16	
Brushes		1.50	
Total			13.66
8139 F.B.M.			
Unit cost per M	$ 1.68		
(3) LABOR:			
Framing, 64 man-hours		48.00	
Erecting, 204 man-hours		144.75	
Painting, 20 man-hours		9.50	
Total			202.25
8139 F.B.M.			
Unit cost per M	$ 24.85		
(4) OTHER COSTS:			
Tool house		$ 14.48	
Bolts		2.95	
Misc. (see distribution under II above)		145.13	
Total			$ 162.56

8139 F.B.M.		
Unit cost per M	$19.96	
Total cost Trestle		$ 867.45
8139 F.B.M.		
38 lin. ft.		
Total unit cost per M.B.M. $106.58		
Total unit cost per lin. ft. trestle $22.89		
VII—APPROACH BACK FILL:		
Labor, 32 team hours, 12 man hours	$ 30.00	
Total		30.00
65 cu. yds. moved:		
Unit cost per cu. yd. $0.46		
VIII—TRUSS:		
(1) STEEL:		
3150 lbs. at $11.90 per cwt.	$374.90	
Freight	8.17	
Hauling at .40 per ton per mile	3.12	
Total		$ 386.19
3150 lbs.		
Unit cost per cwt.	$ 12.26	
(2) LUMBER:		
11045 F.B.M. at $41.00 per M	453.25	
Freight	149.41	
Hauling at .50 per M per mile	75.63	
Unloading at Prineville, 29 man-hours	18.38	
Total		696.67
11045 F.B.M.		
Unit cost per M at bridge	$ 63.08	
(3) PAINT AND CARBOLINEUM	9.15	
Freight	3.16	
Total		12.31
11045 F.B.M.		
Unit cost per M	$1.12	
(4) LABOR:		
Framing, 153 man-hours	132.00	
Erecting, 357 man-hours	264.45	
Painting, 82 man-hours	42.50	
Falsework, 14 man-hours	8.12	
Total		447.07
11045 F.B.M.		
Unit cost per M	$ 40.47	
(5) OTHER COSTS:		
Tool house	30.44	
Framing yard 45 man-hours	25.75	
Misc. (see distribution under II above)	305.23	
Total		361.42
11045 F.B.M.		
Unit cost per M	$ 32.72	
Total cost of lumber in place, per M	137.39	
Total cost of truss (including steel) $1,903.66		

RECAPITULATION OF COSTS

	Unit	Quant.	Cost per unit	Total cost	Total cost per unit	Total cost
(1) Excavation	C.Y.	100	...	.....	8.12	812.27
(2) Concrete	C.Y.	55.03	...	.....	24.78	1363.35
(a) Sand and gravel..........	C.Y.	80	3.84	307.26		
	C.Y.	55.03	5.58			
(b) Cement	Bbl.	55	5.43	298.90		
	C.Y.	55.03	5.43			
(c) Mixing and placing........	C.Y.	55.03	2.17	119.64		
(d) Forms	C.Y.	55.03	5.73	315.17		
(e) Concrete plant	C.Y.	55.03	1.62	89.17		
(f) Finishing	C.Y.	55.03	.14	8.00		
(g) Misc.	C.Y.	55.03	4.09	225.21		
(3) Reinforcing steel	lb.	1250	....	....	0.088	109.75
(a) F. O. B. Portland	lb.	1250	.0557	69.55		
(b) Freight and hauling.......	lb.	1250	.0129	16.07		
(c) Placing	lb.	1250	.0048	6.00		
(d) Misc.	lb.	1250	.0146	18.13		
(4) Approach back fill						
(a) Labor	C.Y.	65	...	.....	.46	30.00
(5) Trestle	M.B.M.	8.139	...	.....	106.58	867.45
	Lin. ft.	38	...	....	22.89	
(a) Lumber	M.B.M.	8.139	60.08	488.98		
(b) Paint and carbolineum.....	M.B.M.	8.139	1.68	13.66		
(c) Labor	M.B.M.	8.139	24.85	202.25		
(d) Misc.	M.B.M.	8.139	19.97	162.56		
(6) Truss		...	....	....	1903.66	1903.66
(a) Steel	Cwt.	31.5	12.26	386.19		
(b) Lumber	M.B.M.	11.045	63.08	696.67		
(c) Paint and carbolineum....	M.B.M.	11.045	1.12	12.31		
(d) Labor	M.B.M.	11.045	40.47	447.07		
(e) Misc.	M.B.M.	11.045	32.72	361.42		
Total cost of lumber in place..	M.B.M.	11.045	137.39	1517.97		

TOTAL COST OF BRIDGE..........................$5086.48

REPORT NO. 2

DETAILED COST REPORT

State Force Bridge Construction

Bridge No. 528

GENERAL DATA:

This bridge was built across the Dry Bed of Crooked River 12½ miles west of Prineville, on the McKenzie River Highway. This river bed is a fissure in the lava about 45 feet wide and 20 feet deep at the bridge site. There is very little soil on the rock, giving an excellent foundation with very little excavation.

The material moved consisted of what little soil there was on top of the rock and that involved in barring out the loose rock and boulders on the east

side to get clearance for the truss. The 30 cubic yards of material were moved in 199 man-hours or 6.63 man-hours per cubic yard, which included the drilling of several holes where the rock was shot off to give better foundation for the pedestals.

There was no suitable material for concrete near the bridge. Gravel and sand was obtained from the road contractors who were graveling the highway near Redmond. It was delivered at an average price of $2.69 per cubic yard. The cement used was from the carload sent to Prineville for bridge No. 540. As the quantity of concrete was small, it was not thought advisable to bring in a mixer. Hand mixing was done, care being taken to get a thorough mix. Form work was simple, as the truss was supported by pedestals. All form lumber was used at least twice and some of it three times. The pedestals for the approach bent on the east side set on solid rock. On the west side the pedestals were placed on the rock fill. The crevices in the fill were filled with concrete under the pedestals and for about two feet outside, so as to give greater bearing. The pedestals supporting the truss are exposed to view and were finished with a carborundum stone.

The forms were built in 89 man-hours or 6.36 man-hours per cubic yard of concrete. By re-using the form lumber 64 F.B.M. were sufficient per cubic yard of concrete. The mixing and placing necessitated 139 man-hours or 13.5 man-hours per cubic yard of concrete.

The approaches consisted of one 19 foot span on each end of the truss. The lumber actually entering into the construction of these totaled 12,309 F.B.M. It required 232.6 man-hours to frame and erect this or 18.70 man-hours per M.B.M. and 108 man-hours or 8.77 man-hours per M.B.M. to paint and brush coat with creosote oil.

The deck lumber was used for falsework, there being two bents at 18-foot centers built of 6x8's and braced with 2x6's. The falsework was built ½ inch low, leaving it free when the rods were tightened up.

The lumber in the truss totaled 22,784 F.B.M. It was framed in 294 man-hours, or 12.90 man-hours per M.B.M. and was erected in 736 man-hours or 32.20 man-hours per M.B.M. The painting required 221 man-hours or 9.69 man-hours per M.B.M. There was one hot brush coat of creosote oil applied up to the top of the felloeguard and above that two coats of standard white woodwork paint were used.

Construction was started February 1st, but had to be shut down February 10th on account of delay in the shipment of materials. Work was resumed on March 19th and the bridge was completed April 21st.

The wages paid on this job were: laborers $4.50 and carpenters $7.00 up to May 1st. After that time laborers received $3.50 and carpenters $6.00.

The detailed costs are as follows:

I—GENERAL EQUIPMENT COSTS:		
Inventory value of equipment transferred to this bridge	$233.70	
Equipment purchased	11.10	
Rentals paid out for equipment rented	30.00	
Total	$274.80	
Less inventory value of equipment transferred from this bridge at end of job	241.80	
Net general equipment charge		$ 33.00

II—ENGINEERING, SUPERVISION AND GENERAL MISCELLANEOUS OVERHEAD EXPENSE:

Expense Ford touring car No. 40	$ 86.38	
Expense Ford touring car No. 46	76.45	
Salary and expense accounts (sup't)	295.54	
Total cost of eng'r, supervision, etc.		458.37
Total equipment, plus overhead (I and II)		$ 491.37

DISTRIBUTION OF ITEMS I AND II:

Excavation	$ 14.74
Concrete (Class B)	34.40
Trestle	103.18
Truss	339.05
	$491.37

III—EXCAVATION (Loose and solid rock):

Labor, 199 man-hours	$140.31	
Powder, caps and fuse	2.55	
Misc. (see distribution under item II)	14.74	
		$157.60

30 cu. yds.

Unit cost per cu. yd. $5.25

IV—CONCRETE:

(1) MATERIALS:

Sand and gravel, 16 cu. yds. at $2.69		43.00	
Cement, 14 bbls. at $3.00		42.00	
Freight on cement, 14 bbls. at $1.94		27.16	
Hauling cement, 14 bbls. at $0.714		10.00	
			$122.16
14 cu. yds. concrete placed: Unit cost for materials per cu. yd.	$ 8.72		

(2) FORMS:

Lumber, 896 feet		$ 29.81	
Hauling lumber		4.40	
Nails		1.90	
Labor, 89 man-hours		74.88	
			110.99
14 cu. yds. concrete placed: Unit cost per cu. yd. for forms	$ 7.93		

(3) MIXING AND PLACING:

Labor, 139 man-hours		70.73	70.73
14 cu. yds. concrete placed: Unit cost for mixing and placing per cu. yd.	$ 5.05		

(4) FINISHING CONCRETE:

Labor, 16 man-hours		10.88	10.88
14 cu. yds. concrete placed: Unit cost for finishing per cu. yd.	$ 0.78		

(5) MISC. EXPENSE (See distribution under II)

Misc. expense		34.40	34.40
14 cu. yds. concrete placed: Unit cost per cu. yd.	$ 2.46		
Total cost of concrete			$ 349.16

Total cost per cu. yd. concrete $24.94

V—TRESTLE:			
(1) MATERIALS:			
Lumber, 12,309 F.B.M. at $41.00 per M		$504.68	
Freight and hauling		182.68	
Unloading lumber at Redmond, 1.78 per M.B.M.		21.88	
			709.24
12,309 F.B.M.			
Unit cost per M for lumber at bridge site		$ 57.62	
(2) PAINT AND CREOSOTE OIL		9.65	9.65
12,309 F.B.M.			
Unit cost per M for paint and creosote oil	$ 0.78		
(3) LABOR:			
Framing, 36 man-hours		31.75	
Erecting, 194 man-hours		149.00	
Painting, 108 man-hours		55.02	
			235.77
12,309 F.B.M.			
Unit labor cost per M	$ 19.15		
(4) OTHER COSTS:			
Tool house		12.68	
Misc. (see distribution under item II)		103.18	
			115.86
12,309 F.B.M.			
Unit cost per M for Misc. expense	$ 9.41		
Total cost of trestle			$1,070.52
Total cost per M.B.M. in place $86.97			
Cost per lin. ft. (38 lin. ft.) $28.17			
VI—TRUSS:			
(1) STEEL:			
Material, 7192 lb. at $9.30 per cwt.		668.55	
Freight and hauling		19.99	
			$ 688.54
7192 lbs.			
Unit cost per cwt.		$ 9.57	
(2) LUMBER:			
Material, 22,784 F.B.M. at $41.00 per M		934.14	
Freight and hauling		347.02	
Unloading at Redmond at $1.78 per M.B.M.		40.62	
Cost at bridge site			1,321.78
22,784 F.B.M.			
Unit cost lumber at bridge site per M	$ 58.01		
(3) PAINT AND CREOSOTE OIL:			
Material, 52 gal. at $0.75		39.20	
Freight and hauling		8.47	
Cost at bridge site			47.67
Cost per M.B.M. of lumber	$ 2.09		
(4) LABOR:			
Framing, 294 man-hours		259.52	
Erecting, 736 man-hours		550.62	
Painting, 221 man-hours		108.92	
			919.06
Unit cost per M.B.M. of lumber	$ 40.34		

(5) Other costs:

Tool house	37.25	
Hauling tools and equipment	28.00	
Framing yard	45.64	
Falsework	45.00	
Misc.	339.05	
		494.94

Unit cost per M.B.M. of lumber.....$21.72

Total cost of truss $3,471.99

Total unit cost of truss=$152.39 per M.B.M.

RECAPITULATION OF UNIT COSTS

	Unit	Quant.	Cost per unit	Total cost	Total cost per unit	Total cost
(1) Excavation	C.Y.	30	...	....	$ 5.25	$ 157.60
(2) Concrete	C.Y.	14	...	....	24.94	349.16
(a) Materials	C.Y.	14	$8.72	$122.16		
(b) Forms	C.Y.	14	7.93	110.99		
(c) Mixing and placing	C.Y.	14	5.05	70.73		
(d) Finishing	C.Y.	14	.78	10.88		
(e) Miscellaneous	C.Y.	14	2.46	34.40		
(3) Trestle	M.B.M.	11.896	...	....	90.00	1,070.52
Trestle	Lin. ft.	38	...	....	28.17	
(a) Materials	M.B.M.	11.896	60.43	718.89		
(b) Labor	M.B.M.	11.896	19.82	285.77		
(c) Miscellaneous	M.B.M.	11.896	9.75	115.86		
(4) Truss		....	...	....	3,471.99	3,471.99
(a) Steel	Cwt.	71.92	9.57	688.54		
(b) Lumber	M.B.M.	22.784	58.01	1,321.78		
(c) Paint and carbolineum	M.B.M.	22.784	2.09	47.67		
(d) Labor	M.B.M.	22.784	40.34	919.06		
(e) Miscellaneous	M.B.M.	22.784	21.72	494.94		
Total cost of lumber			122.16	2,783.45		
TOTAL COST OF BRIDGE						$5,049.27

In studying the above reports it must be borne in mind that the costs although consistent are about 12% higher than present costs (these jobs being completed several years ago) also, the general nature and locality was such as to militate against low costs and the construction was not carried on in seasonable working weather. These same construction crews under identical supervision are doing this character of work at unit costs 25% lower than those shown herein.

The cost reports are of value, however, as indicating the general method of collecting, assembling and reporting such data.

Article 19.—Conclusion

The discussion, which forms the subject matter of this chapter is not presented with an idea of submitting accurate and thoroughly reliable cost data for highway bridge construction. In fact, such data vary from year to year with changing prices, vary with the season of the year during which the work is prosecuted, vary with the locality, the haul, the type of foundation conditions encountered, the organizational excellence of the outfit doing the work, etc., so that no such thing as reliable cost data applicable to any and every case can ever be secured. The data given hereinabove, however, may be said to be roughly indicative of general averages and may serve as a basis for preliminary estimates, which is indeed its immediate and sole purpose.

CHAPTER VII

ILLUSTRATIVE PROBLEM IN TYPE SELECTION

Article 1.—Introductory

In order to illustrate the application of the principles which form the subject matter of the foregoing chapters, a practical problem in type selection will now be considered. The problem chosen is comparatively simple, in fact much simpler than any likely to be encountered in practice. This has been done in order to obviate the necessity for lengthy and detailed discussion and to confine attention to the application of fundamental principles only.

The stream crossing chosen is indicated in profile by Fig. 1. To simplify matters and for the other reasons stated in the foregoing paragraph let it be assumed that the location is already definitely fixed and that the grade line as indicated by Fig. 1 is fixed by considerations outside the scope of this problem. In order to further simplify and shorten the discussion let it be assumed that a 20-foot roadway without sidewalks is desired.

All of these assumptions are made solely for the purpose of shortening this discussion and it must be remembered that in practice a complete type selection study would involve a study of various grade line treatments, of various alignment schemes or location treatments and a study of traffic as regards adequacy of roadway widths.

A 20-foot roadway is rather narrower than desirable for main highway structures but has been selected for this problem in order to permit

a comparison between timber and steel truss construction. Had a 27-foot or even a 24-foot roadway been necessary the timber truss types would be automatically eliminated or nearly so on account of the excessive dimensions necessary for the long timber floor beams involved.

Let it be further assumed that the necessary field surveys have been made and the necessary survey data submitted in the form of maps and profiles (which need not be reproduced here except the cross sectional

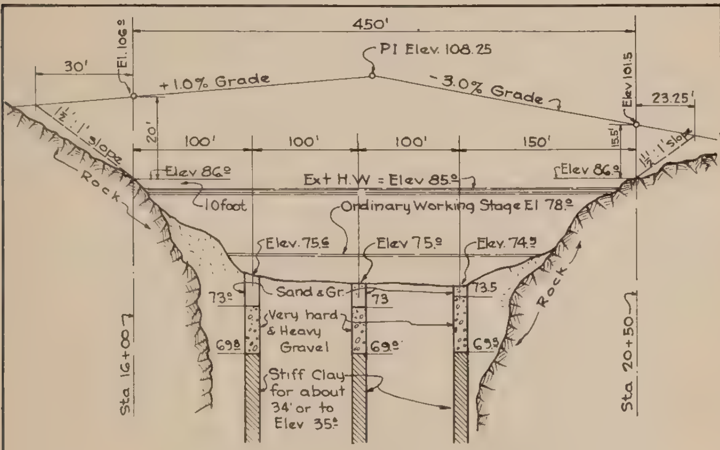


FIGURE 1

stream profile of Fig. 1) and also a tabulation of pertinent cost data from which the following is a summarized extract of the material needed for our problem:

MATERIALS SURVEY

Material	Estimated cost F. O. B. site
Cement	$4.00 per bbl.
Fine aggregate	1.25 per cu. yd.
Coarse aggregate	1.15 per cu. yd.
Structural timber (average)	35.00 per M.B.M.
Piling	0.16 per lin. ft.
Structural steel	.05 per lb.
Reinforcing steel	.03¾ per lb.

A study of Fig. 1 indicates a main stream crossing 450 feet wide with rock out cropping at either shore but dipping sharply toward the channel from both sides; in other words a typical fissure gorge with no flood plain formation on either side. The field cost data submitted also indi-

cates the general range of prices for the various structural materials to be employed from which to estimate unit prices.

Fig. 1 also shows a graphical log of borings at three points in the stream bed from a study of which it may be concluded that stream piers must be carried to about elevation 67 or 68 and founded upon piling, these data also indicate a probable length of piling below the bottom of stream pier footings of about 32 feet or a total length below cut off of about 35 feet. There appears to be very little likelihood of bed erosion as the strata from elevation 73.0 to elevation 69.0 appears quite resistant and it will be further assumed that the stream has cut to permanent grade at this point for which reason the elevation assumed above for the bottom of footings seems ample, especially for a pile foundation driven to elevation 35.0 thereunder.

With these data before us the question of economic analysis and type selection may now be considered. The problem which presents itself appears to be three-fold, as follows:

First:—A selection of the most economical and satisfactory treatment for the end supports (at stations 16+00 and 20+50).

Second:—A selection of the most economical and satisfactory type of construction for the main channel structure (the 450 feet between stations 16+00 and 20+50).

Third:—The determination of the economic span length for the type selected.

Article 2.—Approach Treatments

Regarding the first problem it is observed that there are four methods of treatment for the end supports as follows:

(a) Reinforced concrete abutments and wing walls at stations 16+00 and 20+50.

(b) Mass concrete gravity section abutments and wing walls.

(c) Concrete piers or columns and short concrete approach spans running back from station 16+00 and 20+50, a sufficient distance to provide for a natural slope for the approach embankments at either end. In the present case the slope distance at station 16+00 will be 1½ times 20 feet or 30 feet and that at station 20+50 will be 1½ times 15.5 feet or 23.25 feet. These will require approach spans of about 34 feet and 28 feet respectively to properly retain the fill.

(d) Same treatment as the last arrangement above except that timber trestle spans are used to replace the concrete approach construction.

All of these approach types will be practically the same regardless of

the type of main structure selected except that the through truss designs will require slightly heavier piers or columns at stations 16+00 and 20+50 than either the deck truss or arch types. The reason for this lies in the fact that the forward columns for the through truss designs must support both the approach span girder reaction and the end reaction from the truss while in the deck truss and arch types the end reaction is carried directly into the rock below at the base of these columns. The difference in the quantity of materials however, is comparatively small so that for the present it may be assumed that the end treatments will be more or less independent of the type of main structure selected. This selection, however, is not altogether independent of the selection made for the main span by any means. For example, if timber truss construction should prove economical for the main structure it would hardly be congruous or fitting to adopt a reinforced concrete approach span treatment. Furthermore, the end treatments are important but not in any sense controlling, the main channel treatment being the all important consideration. For these reasons therefore the question of selection of type for the end or approach treatments may be dropped for the present and the investigation may pass on to a consideration of the main channel structure. After the main structural type has been selected an approach treatment which fits most completely and economically into the general scheme may be worked out.

For the present therefore we will eliminate the end supports and consider the main structure as composed of the following elements:

(a) 450 lineal feet of superstructure.
(b) The necessary channel piers.

Article 3.—Type Selection for Main Channel Structure (General)

There are five principal types of construction which may be considered for the main structure as follows:

Scheme A. Reinforced Concrete Arch Construction.
Scheme B. Steel Deck Truss Construction.
Scheme C. Steel Through Truss Construction.
Scheme D. Unhoused Timber Truss Construction.
Scheme E. Housed Timber Truss Construction.

The last two schemes may also be subdivided into deck and through types and furthermore the steel truss types may also be considered in connection with both a reinforced concrete and a timber deck. This matter will be discussed in more detail later on.

Article 4.—Unit Costs

At this point, and before proceeding with the development of quantity data from the curves of Chapter IV it may be well to formulate an estimate as to probable unit costs for the various elements involved. These data may be obtained from the tables of Chapter VI modified and revised in the light of the materials data for this specific problem, as given in the table of Article 1 above.

The unit cost items involved and the assumptions which may be made for the various types are as follows:

Scheme A. Reinforced Concrete Arch Type.

Class A concrete	$25.00 per cu. yd.
Metal reinforcement	.04½ per lb.
Reinf. Conc. hand railing	4.00 per lin. ft.
Hydraulic seal concrete	12.00 per cu. yd.
Wet excavation	4.00 per cu. yd.
Sheet pile cofferdams	0.65 per lin. ft.
Foundation piling	0.50 per lin. ft.
Class D concrete (decks)	25.00 per cu. yd.

Schemes B and C.

Same as above except Class A concrete which will be assumed at ....$22.00 per cu. yd.

In addition

Structural steel (in place and painted)... 0.07 per lb.

Schemes D and E.

Same as for Scheme A except Class A concrete which will be assumed at......$22.00 per cu. yd.

Lumber and timber for trusses (in place).... 80.00 per M.

In connection with the above assumed unit costs the following observations are perhaps in order.

Class A Concrete for Arch Type.—Table 5 of Chapter VI indicates a range in unit cost of from $19.21 to $26.61 for this class of concrete. In the present instance the cement cost is about equal to the average shown on Table 5, while the coarse and fine aggregates are available at the site for a unit cost somewhat less than any shown in Table 5. On the other hand the form work and finishing for the arch type is rather more expensive than the average indicated by Table 5, so that $25.00 seems a fair preliminary assumption.

Metal Reinforcement.—From Chapter VI the unit cost per cwt. for placing reinforcement will average around $0.75 or ¾ cents per pound. Adding this to the F. O. B. price (3¾c) as given heretofore for this particular problem we arrive at the value of 4½c adopted herein.

Hydraulic Seal Concrete.—The value of $12.00 assumed was determined by increasing the cement costs and deducting the cost of forms and of finishing as discussed under Article 7 of Chapter VI.

Wet Excavation. — The price of $4.00 assumed is about in accordance with the discussion in Article 3 of Chapter VI. Perhaps the value $4.00 is somewhat high in view of the fact that cofferdam work is to be figured separately. However, the character of the material to be moved makes digging rather difficult (the material below elevation 73.0 being rather hard), also the yardage involved will be small. All in all, therefore, perhaps the $4.00 per cu. yard price should stand, for the present purpose at least.

Sheet Pile Cofferdams.—It is an open question whether sheet piling or timber crib work should be used for the stream piers for this job. It looks off hand like a sheet pile job with a 9-inch "Wakefield" type which has been estimated at 65c per foot in place in accordance with Article 3 of Chapter VI.

Foundation Piling.—There is no way of determining the number of lineal feet of foundation piling needed until it is determined whether one or two or three stream piers are to be used. Assuming that over 2,000 lineal feet and less than 10,000 lineal feet will be used (which is a safe assumption) Article 13 of Chapter VI indicates an average price of about 30c per foot for driving which added to the F. O. B. price given hereinabove results in a unit cost of about 50c per foot in place, after adding an arbitrary item of 4c per foot to allow for cut offs. This is the value used.

Class D Concrete.—The cost of cement for this class of concrete is increased over and above that for Class A concrete, on the other hand the cost of form work and finishing will be less. There will not be a great deal of difference in cost so that for convenience the same value ($25.00 per cu. yd.) may be employed for both.

Class A Concrete for the Piers of the Steel and Timber Designs.—The Class A concrete for these piers is, of course, no less expensive than that for the piers of the arch design, however in the arch design the Class A concrete also includes the ribs, braces, spandrel columns, etc., wherein the form work and placing costs run rather high. The average of $25.00 per cu. yd. for the arch type includes this expensive work, whereas no such work is included for the steel and timber designs. The average form and placement costs, therefore, may be assumed to be decreased about $3.00 per cu. yd. making a net total cost of $22.00 per cu. yd. which is the value assumed.

At this point it might be well to observe that Class B concrete might be used for pier footings and perhaps for shafts but since the resulting decrease in cement cost is small and in order to simplify the problem Class A concrete has been assumed as being used throughout.

Structural Steel.—From Article 11 of Chapter VI a unit cost of 7c per lb. appears about correct where the F. O. B. cost amounts to 5c per lb. as is the case here.

Lumber and Timber for Truss Construction.—From Article 12 of Chapter VI it will be observed that there is a great deal of variation in cost between timber superstructures proper (decks and stringers) and truss construction. In this case, however, the bulk of the timber work will be in the trusses and housing and in view of the fact that unit costs for this class of work are at the best quite variant, and in order to simplify the discussion it may be safely assumed that 25% of the timber used will be framed and erected at a cost given in Chapter VI as the average for superstructure work ($17.50) while 75% of the timber will involve an expense for framing equal to the average given in Chapter VI for truss construction.

Applying these coefficients

$$\begin{aligned} \$17.50 \times 25\% &= \$\ 4.37 \\ 54.00 \times 75\% &= \ \ 40.50 \\ \hline & \ \ \$44.87 \end{aligned}$$

The average cost of the timber F. O. B. the job (from Article 1) amounts to about $35.00 per thousand, making a total of $79.87 per M.B.M. in place. The assumed unit cost of $80 per M in place, therefore, if applied to all lumber in the job including deck and stringers is not far from a correct value.

In addition to the unit costs hereinabove set forth there is, of course, an arbitrary percentage (about 3%) to add for miscellaneous and minor items, and also a percentage for general equipment costs, a percentage for general overhead expense and one for contractor's profit. As none of these affect the comparative cost values the same can well be omitted from the discussion until the naked cost of the various types has been compared and the final selection made.

Article 5.—Discussion of Scheme A (Reinforced Concrete Arch Type) and Determination of Economic Span Length for Same

We are now ready to consider Scheme A (the reinforced concrete arch type) and to determine the economic span length for the same.

It is quite apparent that the selection of span lengths resolves itself into a choice between the following arrangements:

1—One span at 450 ft.
2—Two spans at 225 ft.
3—Three spans at 150 ft.
4—Four spans at 112½ ft.

5—Five spans at 90 ft.

6—Six spans at 75 ft.

The first arrangement may be eliminated from consideration without further thought, as the cost of a single span of this length with the rise limited as indicated by Fig. 1 would be abnormally excessive in reinforced concrete.

The first step in considering the other arrangements is the determination of the cost of one stream pier.

At the very outset it must, of course, be remembered that the function of an arch pier is dual; first, to support the dead load reaction, which is vertical and balanced, and second to support the unbalanced live load thrust from a loaded span on one side only. The size of the river piers are, therefore, more or less dependent upon the length of the adjacent spans and thus at the very outset we run into a complication for the reason that the arches are not yet analyzed and the pier dimensions cannot be determined exactly. A very close approximation may be made, however, by sketching in the probable dimensions for a river pier for the two span design and one for the six span design and interpolating between.

This procedure has been followed in Figures 2 and 3 from which the following results are derived:

Cost of one pier:

Two span design	$6,603.00
Three span design	6,295.00
Four span design	5,987.00
Five span design	5,679.00
Six span design	5,371.00

It will be observed that in computing the cost of a stream pier the area of the cofferdam was taken as one foot larger all around than the neat lines of the footing masonry. The sheet pile cofferdam was cut off 3 feet above ordinary water and driven to an elevation 6 feet below footing level and well into the clay. An hydraulic seal was used for estimating purposes, the total thickness being assumed as 5 feet, as a matter of fact with a 6 foot cofferdam seal into stiff clay of this kind it might be possible to pump the foundation pits and place the footings in the dry. This procedure, however, would only save two feet at the most, as the pier footing should be at least 3 feet in thickness in order to properly distribute the reactions from the superstructure into the piling, so that at most the average cost for each pier would only be reduced by about $380.00. The full theoretical seal depth necessary for an hydraulic seal of this kind is 42/100 of the working head or in this case 42/100 of

(78-67) or 4.62 feet to which should be added a "leveling up course" deposited in the dry of not less than 18 inches thickness making a total theoretical depth needed of about 6 feet. In view, however, of the impermeability of the clay stratum and of the holding value of the piling this figure may safely be cut to 5 feet (as was done in Figures 2 and 3) depositing the first 3.5 feet as an hydraulic seal proper and the last 1.5 feet as a leveling up course in the dry after the cofferdam has been unwatered.

Figures 2 and 3 will also disclose the fact that the pier selected is of the round nosed type with cellular construction between the arch ribs. The piling were assumed as cut off at an elevation 3 feet above footing elevation, leaving a distribution slab 2 feet in thickness, all of which represents good conservative practice.

With these data (as to pier costs) we are now ready to proceed with the selection of the economic span length.

From Figs. 102, 103 and 104 of Chapter IV the following data is obtained for each arrangement of arch spans. The average height *H* (Fig. 102) will be about 11.0 feet for all span arrangements.

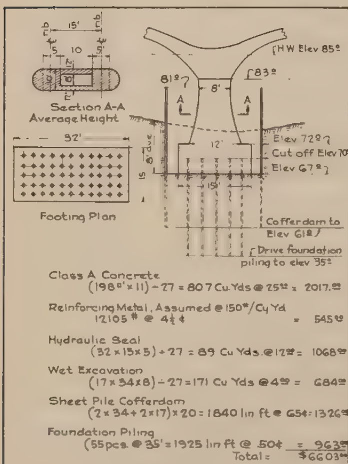


Figure 2
Sketch and Estimate of River Pier
For Arch Type (2 Spans)

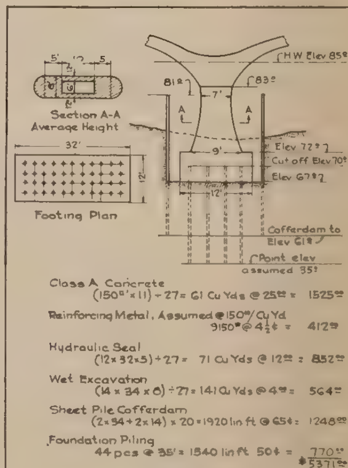


Figure 3
Sketch and Estimate of River Pier
for Arch Type (6 Spans)

NO. OF SPANS	QUANTITIES PER LINEAL FOOT OF BRIDGE			
	DECK, SPANDREL ARCHES AND COLUMNS		ARCH RIBS AND BRACING	
	CONCRETE C.Y.	REINFORCING LB.	CONCRETE C.Y.	REINFORCING LB.
2	1.03	179	3.20	800
3	1.03	179	1.68	462
4	1.03	179	1.18	303
5	1.03	179	0.92	218
6	1.03	179	0.83	162

In addition to the above, for each design, there will be needed two rather small end abutments. These abutments may be estimated by means of a rough sketch as indicated in Fig. 4 from which it is observed that about 75 cu. yds. of concrete will be needed for the two abutments.

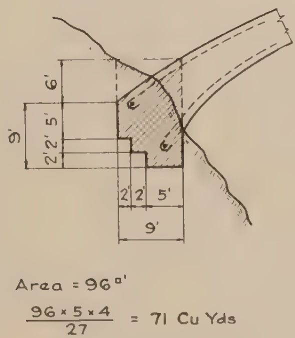


Figure 4

It might become necessary to tie the two individual rib abutments together by means of a heavy strut or foundation mat at each end. However, the additional cost would be rather small and may be neglected for preliminary comparisons. These abutments will not vary materially with the span length of the arch springing therefrom as they are really only bedding seats set into the solid rock. In addition to the concrete there will be required a small amount of reinforcing metal (say 100 lb. per cubic yard of concrete or 7,500 lb. for the entire bridge).

We are now ready to make the final comparison as follows:

Cost item	2 spans	3 spans	4 spans	5 spans	6 spans
Hand Rail:					
900 lin. ft. at $4.00......	$ 3,600.00	$ 3,600.00	$ 3,600.00	$ 3,600.00	$ 3,600.00
Two Abutments:					
75 cu. yds. conc. at $25.00	1,875.00	1,875.00	1,875.00	1,875.00	1,875.00
7500 lb. reinf. at 4½c...	338.00	338.00	338.00	338.00	338.00
Deck, Spandrel Columns, etc.:					
Conc., 450x1.03 = 463.5 cu. yds. 463.5 cu. yds. at $25.00	11,588.00	11,588.00	11,588.00	11,588.00	11,588.00
Reinf., 450x179 = 80550 lb. 80550 lb. at 4½c.	3,625.00	3,625.00	3,625.00	3,625.00	3,625.00
River Piers	6,603.00	12,590.00	17,961.00	22,716.00	26,855.00
Arch Ribs and Bracing:					
Conc., 450x3.20 = 1440 cu. yds. at $25.00.....	36,000.00				
Reinf., 450x800 lb. = 360,000 lb. at 4½c....	16,200.00				
Conc., 450x1.68 = 756 cu. yds. at $25.00........		18,900.00			
Reinf., 450x462 lb. = 207900 lb. at 4½c		9,356.00			
Conc., 450x1.18 = 531 cu. yds. at $25.00........			13,275.00		
Reinf., 450x303 lb. = 136350 lb. at 4½c.....			6,136.00		
Conc., 450x0.92 = 414 cu. yds. at $25.00........				10,350.00	
Reinf., 450x218 lb. = 98100 lb. at 4½c......				4,415.00	
Conc., 450x0.83 = 374 cu. yds. at $25.00........					9,350.00
Reinf., 450x1.63 = 72900 lb. at 4½c.......					3,280.00
Total	$79,829.00	$61,872.00	$58,398.00	$58,507.00	$60,511.00

From the above comparison it will be noted that the 4-span structure (4 spans at 112 feet 6 inches each) appears to be the economic arrangement and that the total naked cost amounts to $58,398.00.

It should be observed in passing that the 2-span structure (225-foot span lengths) falls outside the limits for the curves given in Figs. 103 and 104 of Chapter IV so that the values given hereinabove have, of necessity, been obtained by extrapolation. The rise span ratios for the two span and also for the three span arrangements are less than conservative practice would dictate and likewise fall without the limits of the curves of Figs. 103 and 104, which figures have been limited to

include only those span lengths and rise-span ratios which experience has indicated to be conservative and economical.

The results of the comparison above set forth are sufficient in themselves to indicate that under ordinary conditions a reinforced concrete arch span in excess of 200 feet or with a rise-span ratio of less than about 0.15 will rarely prove to be an economical construction type.

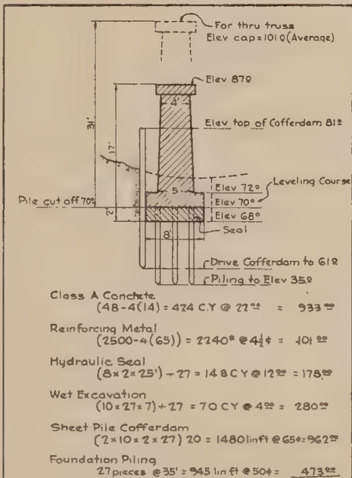


Figure 5
Sketch and Estimate
of River Pier for Truss Types

Article 6.—Discussion of Scheme B (Steel Deck Truss Construction) and Determination of the Economic Span Length for Same

We now come to a consideration of the next construction type; viz., the steel deck truss span with concrete piers, roadway, and railing.

We will first compute the cost of one stream pier as in the case of reinforced concrete arch design. In this case the foundations need not be carried quite so deep as in the case of arch design, as the footing pressure is all vertical. Let us assume a footing elevation of 68.0. Figure 123 of Chapter IV indicates the general dimensions for dumbbell piers of the type which will be adopted here. The quantity curves of Figure 128 give the quantities of concrete and reinforcing metal for piers of this type for varying heights including a footing two feet in thickness. In the present instance we will add an hydraulic seal two feet in total thickness. During the construction the first three feet would

doubtless be poured as a seal and the last foot as a leveling up course but for convenience we have split the thickness up in the manner shown on the sketch of Figure 5 (2 feet of seal and 2 feet of footing) this being merely for convenience in using the curves. There is not much question but that a 3-foot seal will hold. The full theoretical thickness is less than 5 feet and the frictional resistance of the piling which can be safely assumed at 5 pounds per square inch of pile surface (exclusive of the head area) will develop a resistance equivalent to an additional seal thickness of several feet alone. The impermeability of the clay strata may render pumping the foundation pits and depositing the entire pier in the dry a possibility. This would reduce the cost of each pier by $178.00 but since the resulting footing would be only two feet thick which is rather shallow, the estimate as it has been made should probably not be reduced.

The distance center to center of pier shafts for a through truss design of this roadway is indicated on Figure 128 of Chapter IV to be 21 feet 8 inches. For the deck truss design this dimension may be reduced 4.0 feet on account of the possibility of overhanging the curb and handrailing. This has been done in the estimate shown on Figure 5.

The data as to the weights of structural steel for the various span lengths are obtained from Figure 83 of Chapter IV, as are also the data for the quantities of material in the deck and wheel guard with the exception of the steel for the 450-foot span. This latter item is obtained from Figure 156 by assuming a total load of 3,600 pounds per lineal foot of truss which is not far from correct.

The above data can now be assembled, the unit prices applied and the entire span length comparison tabulated as follows:

SPAN ARRANGEMENT

		1 @ 450		2 @ 225		3 @ 150		4 @ 112.50	
ITEM	RATE	QUAN.	AMOUNT	QUAN.	AMOUNT	QUAN.	AMOUNT	QUAN.	AMOUNT
Structural steel	$0.07	1,125,000	$78,750	656,000	$45,920	504,000	$35,280	480,000	$33,600
Reinf. conc floor:									
Concrete	25.00	234 c.y.	5,850	....	5,850	....	5,850	....	5,850
Reinf. metal	.04½	33,750 lb.	1,519	....	1,519	....	1,519	....	1,519
Stream piers	......	....	....	(1)	2,927	(2)	5,854	(3)	8,781
Reinf. conc. handrail	4.00	900	3,600	900	3,600	900	3,600	900	3,600
Total			$89,719		$59,816		$52,103		$53,350

The above figures include everything but the excavation and concrete work necessary for the end bridge seats. However, as this item is small it may be roughly estimated at 20 yards of concrete and the same amount of rock excavation, amounting roughly to say $600.00.

From the above study we may conclude that the 3-span structure represents about the economic span length for the deck truss construction

and that the total naked cost is about $52,103.00 plus $600.00 or $52,703.00.

ARTICLE 7.—DISSCUSSION OF SCHEME C (STEEL THROUGH TRUSS CONSTRUCTION) AND DETERMINATION OF ECONOMIC SPAN LENTH

Considering next the through truss design the quantities in the stream piers will be greater than for the deck truss type for two reasons.

First. The piers are higher.

Second. The distance from center to center dumbbells will be four feet greater.

From these data and the sketch of Figure 5, we determine the following:

Class A concrete	
78 cu. yds. at $22.00 per cu. yd.	$1,716.00
Reinforcing metal	
4,000 lb. at 4½¢	180.00
Hydraulic seal	
(8x2x29) ÷ 27 = 17.2 cu. yds. at $12.00	206.00
Wet excavation	
(10x31x7) ÷ 27 = 80 cu. yds. at $4.00	320.00
Sheet pile cofferdam	
(2x10+2x31) 20 = 1,640 lin. ft. at 65¢	1,066.00
Foundation piling	
30 pcs. at $35.00 = 1,050 lin. ft. at 50¢	525.00
Total	$4,013.00

Using these data for pier costs, the steel quantities from Figure 71 of Chapter IV, and the quantities for deck and hand railing from Figure 70 of Chapter IV, we obtain the following comparison:

SPAN ARRANGEMENT

		2 @ 225		3 @ 150		4 @ 112.5	
ITEM	RATE	QUAN.	AMOUNT	QUAN.	AMOUNT	QUAN.	AMOUNT
Structural steel	$0.07	620,000 lb.	$43,400	480,000 lb.	$33,600	460,000 lb.	$32,200
Reinf. conc. floor:							
Concrete	25.00	234 cu. yds.	5,850	234 cu. yds.	5,850	234 cu. yds.	5,850
Reinf. metal	.04½	33,750 lb.	1,519	33,750 lb.	1,519	33,750 lb.	1,519
Stream piers		(1)	4,013	(2)	8,026	(3)	12,309
Total			$54,782		$48,995		$51,608

For the through truss type the end piers must be heavier than for the deck truss spans as was pointed out hereinbefore (See Art. 2.) The excess yardage over and above that necessary for carrying the supports for the approach spans must be added therefore to make this type com-

parable with Schemes A and B. Roughly about 25 cubic yards of concrete will be needed for both end supports, also about 2,500 pounds of reinforcing metal, which at the unit prices heretofore assumed amounts to $662. This amount must therefore be added to the totals given hereinabove.

From the above discussion therefore we conclude that the economic span length for the through truss type is 150 feet as was the case for the deck truss design, and that the total naked cost is $48,995 plus $662.00 or $49,657.00.

ARTICLE 8.—DISCUSSION OF SCHEME D (UNHOUSED TIMBER TRUSS CONSTRUCTION) AND DETERMINATION OF THE ECONOMIC SPAN LENGTH

Taking up now the consideration of Scheme D, the unhoused timber truss design, it becomes at once apparent that there will be no need to consider the through truss type. The reason for this is as follows:

The river piers will be practically the same for the timber design as for the steel design, viz:

For deck construction—$2,927.00.

For through construction—$4,013.00.

or a difference of over $1,000.00 in favor of the deck type. Now, in the case of the steel design there was an offsetting credit for the through type of design owing to the elimination of a reinforced concrete handrail. In the case of the timber trusses no such offsetting charge exists, moreover the timber deck type utilizes the traffic deck for a roof and thus dispenses with the necessity for a shingled roof, so that whenever clearance room permits the deck timber truss will prove the cheaper. There are other reasons such as the shortening of the floor beam span due to the possibility of overhanging the deck, etc., all of which favor the deck type. All in all, therefore, the through truss type in timber need not be considered for this problem. (If the clearance were not sufficient to permit the use of deck construction then the through truss type would find a field of application).

In reference to economic span lengths, no consideration need be given to any span length in excess of 150 feet as this is about the limiting length for timber truss construction as was pointed out in Chapter IV. On the other hand it may be at once concluded that since 150 feet has been found to be the economic span length for steel construction, the present arrangement (wherein the piers are exactly the same and the superstructure much cheaper) will not show an economic span length of less.

The arrangement for this scheme therefore resolves itself into the following:

1—2 stream piers at $2,927 or		$ 5,854.00
2—3 150 ft. unhoused timber deck spans, each requiring the following material (see Figure 76, Chapter IV)		
Lumber, 77 M at $80.00	$6,160.00	
*Castings, 23,000 lbs. at 10c	2,300.00	
Structural metal, 36,000 lbs. at 7c	2,520.00	
Total	$10,980.00	
Total for 3 spans		32,940.00
3—2 end bridge seats (same as for Scheme B)		600.00
Grand Total		$39,394.00

*The castings will cost more than the structural metal, 10c per pound in place is probably a fair assumption.

Article 9.—Discussion of Scheme E (Housed Deck Timber Truss Construction)

The cost of this type of construction may be readily obtained from the cost data for Scheme D by adding to each span the cost of housing and of paint for the same.

These data may be obtained from Figures 58 and 47 as follows:

Since Fig. 47 represents the quantities in unhoused trusses and Fig. 58 represents those for housed truss construction the amount of timber in the housing may be roughly obtained by subtraction. In this manner we obtain the following:

Lumber in housing (one span):		
92 M—70 M = 22 M at $80.00	$1,760.00	
Paint and preservative:		
225 gals. at $2.00	450.00	
Total	2,210.00	
Total for 3 spans		$6,630.00
Total cost of unhoused deck truss construction (from Art. 8)		39,394.00
Total cost of housed deck truss construction =		$46,024.00

Article 10.—Steel Truss Designs With Timber Decks

From Figure 70 of Chapter IV we obtain the following quantities for a 6 inch timber deck and handrailing.

(A) Lumber:	
Deck	120 F. B.M.
Handrail	21 F. B.M.
(B) Hardware (deck plus rail)	553 lb.

Estimating the decking lumber at $60.00 per thousand in place, the

hand railing at $80.00 per thousand in place and the hardware at 10¢ per pound, we have the following cost per lineal foot of span for the timber deck:

Deck, 120 ft. B.M. at 6c	$7.20
Handrailing, 21 ft. B.M. at 8c	1.68
Hardware, 553 lb. at 10c	.55
Total	$9.43 per lin. ft.

Total cost of 6 inch timber deck and railing for the main span $= 450 \times 9.43 =$ $4,243.00. Applying this figure to the 150-foot deck truss spans discussed in Article 6 of this chapter we have the following:

Structural steel, 504,000 lb. at 7c	$35,280.00
Stream piers	5,854.00
Timber deck and rail	4,243.00
Additional concrete for bridge seats at end piers (see Art. 6)	600.00
Total	$45,977.00

For the through truss design we have the following:

Structural steel, 480,000 lb. at 7c	$33,600.00
Stream piers	8,026.00
Timber deck and rail	4,243.00
Additional concrete for end piers (see Art. 7)	662.00
	$46,531.00

These last two are nearly a stand-off and if either one is selected some closer figuring will be needed to determine the exact difference in first cost.

Article 11.—Recapitulation of First Costs

Summarizing the results of our analysis to date as regards first costs we may tabulate as follows:

Scheme A.	Reinforced Concrete Arch Type	$58,398.00
Scheme B.	Steel Deck Truss Type with Reinforced Concrete Deck and Rail	52,703.00
Scheme B-1.	Same but with Timber Deck and Rail	45,977.00
Scheme C.	Steel Through Truss Type with Reinforced Concrete Deck	49,657.00
Scheme C-1.	Same but with Timber Deck	46,531.00
Scheme D.	Unhoused Timber Truss	39,394.00
Scheme E.	Housed Timber Truss	46,024.00

These data, however, do not do us a great deal of good; the arch is clearly the most expensive but clearly the more nearly permanent. On the other hand, the unhoused timber structure is by far the cheapest and also by far the least enduring. It is at this point that the principles developed and discussed in Chapters III and V come into play. Figure II of Chapter V is more or less of a summary of results for the entire text

of the chapter and from the coefficients there given the first costs hereinabove tabulated may be converted into annual costs from which a true economic comparison may be made. This operation, which constitutes the final step in the problem, is discussed in the next article.

ARTICLE 12.—APPLICATION OF ECONOMIC CONSTANTS TO THE COST DATA

In order to shorten the discussion we will assume an interest rate of 5% and an insurance rate on the timber of 3/10%. Obviously other rates might be tried out or assumed if conditions demand which would lengthen but not alter the method of analysis.

Let us also assume that the structure is on a main traveled highway, in a populous district where appearances are not to be disregarded. From the discussion which forms the subject matter of Chapter V we are therefore warranted in assuming a certain rental value for the structure as an offsetting expense item. Let us assume a 2% rental value under which assumption the following coefficients,

$$K = (r + \frac{M}{C} + \frac{R}{C} + \frac{I}{C} - P)$$

are obtained:

Concrete construction .... $K = 5.1\%$
Steel construction .... $K = 6.2\%$
Timber construction (housed) .... $K = 9.5\%$
Timber construction (unhoused) .... $K = 11.5\%$

The value $K = 11.5$ is taken from Figure 11 of Chapter V as representing a rough mean between groups I and II of the unhoused timber construction. The exact value to employ in any given case will of course depend upon climatic and other conditions as discussed in Chapter V.

Using the above values, and segregating for each design type the concrete, the steel, and the timber items, we arrive at the following tabulation of results:

Type	Cost	Value of K	Annual cost Partial	Annual cost Total
Type A, R. C. Arch:				
Concrete constr.	$58,398	5.1%	...	$2,978
Type B, Deck Truss:				
Concrete constr.	17,423	5.1%	889	
Steel constr.	35,280	6.2%	2,187	3,076
Type B1, Deck Truss:				
Concrete constr.	6,454	5.1	329	
Steel constr.	35,280	6.2	2,187	
Timber constr.	4,243	11.5	488	3,004

Type C, Through Truss:				
Concrete constr.	16,057	5.1	819	
Steel const.	33,600	6.2	2,083	2,902
Type C1, Through Truss:				
Concrete constr.	8,688	5.1	443	
Steel constr.	33,600	6.2	2,083	
Timber constr.	4,243	11.5	488	3,014
Type D, Unhoused Timber:				
Concrete constr.	6,454	5.1	329	
Timber constr.	32,940	11.5	3,788	4,117
Type E, Housed Timber:				
Concrete constr.	6,454	5.1	329	
Timber constr.	39,570	9.5	3,759	4,088

From the above analysis it is apparent that the timber truss spans are more or less out of the running, the steel spans with timber decks are also eliminated and the deck truss with concrete railing and roadway can not compete with the reinforced concrete arch type. The choice of type, therefore, becomes one between the reinforced concrete arch type at an annual cost of $2,978 as against the through truss type with concrete roadway at an annual expense of $2,902, or $76 per annum cheaper. The arch span affords an unobstructed traffic vision which is not the case with the through truss type. If the approaching alignment is curved the arch type would undoubtedly be warranted and in any event the difference between these two types is so small that the arch type would doubtless be considered a wise selection.

It is not to be expected that the arch type will show marked economy for foundation conditions of this character, and its only chance to compete at all, has resulted from the use of a 2% rental charge or, in other words, from the fact that the maximum possible value has been given to the architectural excellence or aesthetic value of the type.

In order to determine the degree to which we have been influenced by considerations of appearance we may now proceed by recalculating the various annual expense items with the rental charges entirely neglected. The results obtained are as follows:

Type	Cost	Value of K	Annual cost Partial	Annual cost Total*
Type A:				
Reinforced conc. arch, conc. constr.	$58,398	7.1	....	$4,146
Type B:				
Concrete cost	17,423	7.1	$1,237	
Steel construction	35,280	8.2	2,893	4,130

Type B-1:				
Concrete construction	6,454	7.1	458	
Steel construction	35,280	8.2	2,893	
Timber construction	4,243	13.5	572	3,923
Type C:				
Concrete construction	16,057	7.1	1,140	
Steel construction	33,600	8.2	2,755	3,895
Type C-1:				
Concrete construction	8,688	7.1	617	
Steel construction	33,600	8.2	2,755	
Timber construction	4,243	13.5	572	3,944
Type D (unhoused):				
Concrete construction	6,454	7.1	458	
Timber construction	32,940	13.5	4,447	4,905
Type E (housed):				
Concrete construction	6,454	7.1	458	
Timber construction	39,570	11.5	4,550	5,008

*With annual rental charges entirely neglected.

A study of this last tabulation indicates that again the timber spans and also the steel structures with timber decks are practically eliminated. In this latter tabulation, however, the steel deck truss design with concrete roadway and balustrade shows a slight advantage over the arch type and the through truss type with concrete roadway shows a marked economy.

From the above study, therefore, it may be concluded that for conditions rendering it the part of wisdom to adopt a pleasing architectural treatment, the arch type (Type A) should be adopted while for locations wherein this feature is of minor importance the through truss type with concrete deck (Type C) represents the most economic and advantageous selection.

It will be observed that the choice between the main competing types involves very close comparisons in this particular case. In practice, therefore, the selection would probably not be made until some rather close designing had been done and exact quantities calculated for the principal competing types. In fact for a choice as close as this it might prove desirable to prepare plans and take bids on the two best design types in order to determine the exact difference in first cost.

Reference to Figure 11 of Chapter V and to the economic equation shown thereon will remind us of the fact that as yet no consideration has been given to the value O representing the annual expense of traffic operation. The reason that this factor was not considered in deciding upon a type selection was simply because it was not needed. In this particular problem the only factor in the design affecting traffic operation

costs is the type of deck employed and for the principal competing types the deck construction was identical.

If the comparison hereinabove submitted had indicated a saving in annual expense for one of the timber deck types of a certain amount (say $400.00) it would then have been necessary to have determined the present and estimated future traffic density over the structure and to have applied thereto the values given in Chapter III as to unit traffic costs for different deck types.

To illustrate let us assume an average traffic density of 1,000 vehicles per day as a probable near future condition. From Chapter III it will be observed that the minimum difference in traffic costs for the concrete as against the timber deck design will amount to about 2 cents per vehicle-mile. Applying this figure, the saving in traffic operation cost for the concrete deck types amounts to the following:

$$\frac{450}{5280}\text{ miles} \times 365 \text{ days} \times 1{,}000 \text{ cars} \times 2\text{c} = \$622.00 \text{ per year.}$$

which more than offsets the $400.00 saving above assumed so that for a traffic density of 1,000 vehicles per day the concrete deck would still show economy. If, on the other hand, the traffic were to average only 300 cars per day the saving in traffic costs would amount to only 3/10 of the above or about $186.00 and the timber deck would show economy and so on.

The foregoing discussion has probably been carried sufficiently far to illustrate the methods to be employed and the application of the principles which form the subject matter of the foregoing chapters. Other assumptions as to rental values or different assumptions as regards traffic service or conditions regarding decay in timber, etc., might be made but these would serve but to lengthen the problem without illustrating anything new as regards the method of attack for which reason the discussion need not be carried further.

Mention was made in Article 2 of this Chapter of the selection of approach treatment. Having made our selection as regards the main span, this phase of the problem might now be taken up. However, the analysis would be but a repetition of the methods hereinabove illustrated and need not be given here in more detail. It is apparent that having eliminated from consideration the timber deck types, the choice for approach treatment becomes purely one of determining the relative first costs of Types A, B and C as described under Article 2. This may be readily accomplished from the quantity curves of Chapter IV in connection with the unit cost values given in the present chapter and one additional cost unit viz., the cost of approach enbankment per cu. yd. in place.

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